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A REPORT OF A LOADING TEST ON TWO WELDED STEEL TRUSSES

by

J. W. N. FEAD

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PART I

INTRODUCTION

The following report is a resume of testing procedures used during, and the results obtained from, a loading test of two welded steel, truss-type, floor joists. The experimental work covered by this report was conducted during the period, October 1947 to May 1948.

Such incidental tests as were used for the purposes of perfecting techniques are summarized in Appendix III.

NATURE OF INVESTIGATION

The primary purposes of the investigation were:

1. To obtain an evaluation of the total axial stress, or member "load", for each truss member. This value was to be compared with the theoretical value obtained using the theory of simply-connected trusses.

2. To obtain an evaluation of the secondary stresses due to bending in the members.

No attempt was made to evaluate the secondary stresses in the neighborhood of the joints. It would have been impossible to obtain satisfactory results in such a case because of the type of construction encountered in the trusses.

In addition, an overall picture of the performance of these joists was desirable. Such information would provide a basis for decisions as to the suitability of this type of member for different types of buildings.

OUTLINE OF INVESTIGATION

The trusses used in the investigation were fabricated locally. Since they were designed on the basis of an analysis as a simply-connected truss, it was felt that there was a need for a study of the actual conditions under loading.

The author has been unable to discover reports of any similar investigations. In fact, aside from a rather rough test conducted by the manufacturer, this author does not know of any such experiment having been performed. The one test mentioned did show satisfactory load carrying and deflection characteristics for normal building use.

The results of the present investigation gave a satisfactory analysis for total axial stress, (hereinafter referred to as "total stress"), with the exception of the end portions of the top chord.

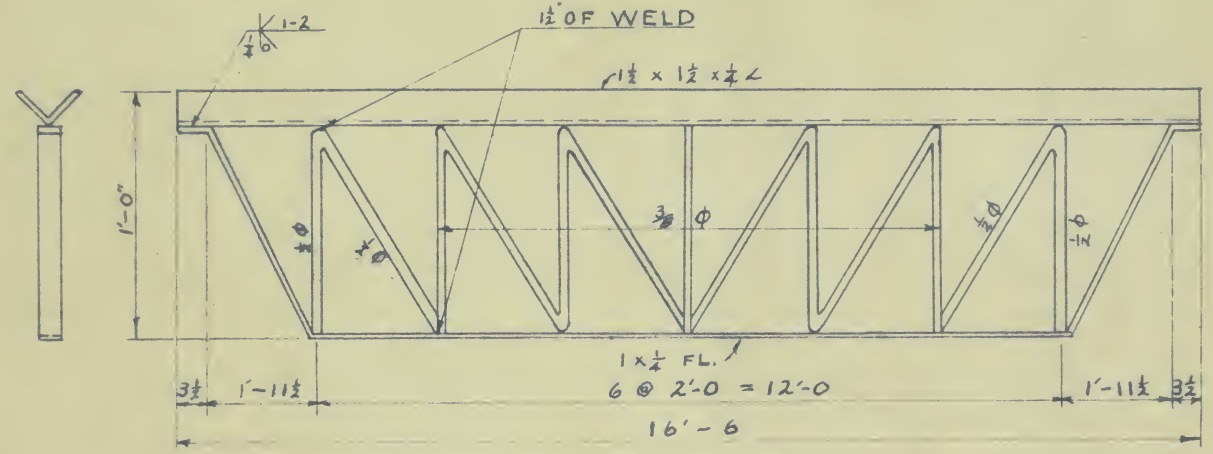
The results showed such a wide variation in bending stresses that little of value could be obtained from them.

The overall performance of the trusses was observed to be satisfactory.

The trusses used were composed of structural steel which had been welded at the joints. The specifications called for the overall length of truss to be sixteen feet, six inches. This allowed a length of three and one-half inches for bearing at each end, two end panels of one foot, eleven and one-half inches , and six interior panels of two feet, zero inches. The sizes of members and their arrangement are clearly shown in Figure 1.

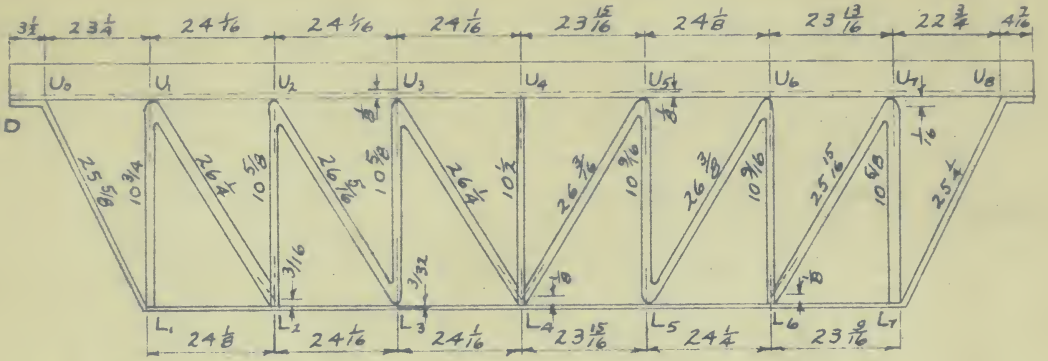
The actual dimensions of the two trusses, as constructed, are shown in Figures 2 and 3.

It is interesting to note that the connections of the web



THEORETICAL TRUSS

FIGURE 1



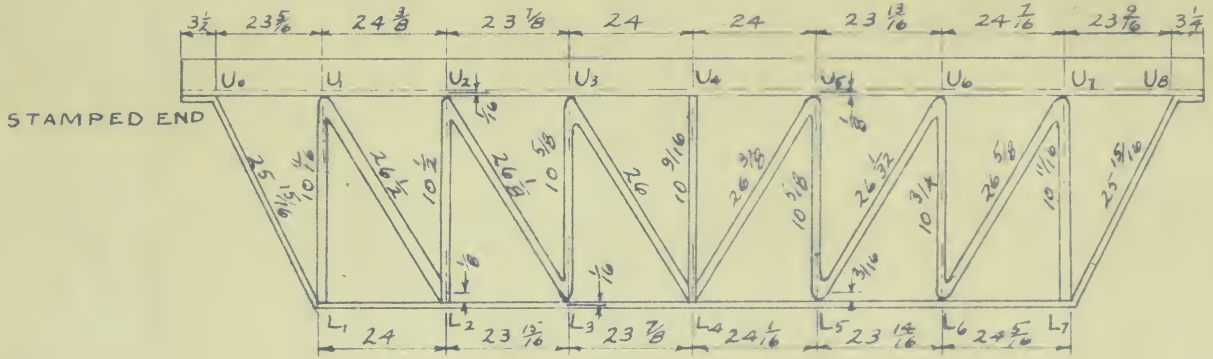
Verticals – measured from bottom of ∠ to top of strap

Diagonals – " " ℄ vertical to ℄ vertical.

Joints – Intersection of ℄'s of vertical and diagonal.

DIMENSIONS – TRUSS NO. 1

FIGURE 2



See notes on Figure 2

DIMENSIONS-TRUSS NO.2

FIGURE 3

members with the upper chord were made at the bottom of the "V" formed by the top chord angle. This meant that the intersection of the vertical and diagonal at any joint was well below the centre of gravity of the top chord. The eccentricity of loading thus introduced bending moments in the chord. In the normal use of these trusses, the loading is applied uniformly to the top chord. Hence, the bending moments introduced by eccentricity helped to offset the bending moments arising directly from the loading.

There were a number of lower chord joints which were also eccentric. The most notable of these were the joints, L_1 and L_7 in each truss. The bending moments arising at these lower chord joints served only to aggravate the stress conditions in the members, thus lowering the carrying capacity of the trusses.

As shown in Figure 1, in each truss, the members, U_2-L_2, U_2-L_3, U_3-L_3, and U_3-L_4, were composed of a single piece of three-eighth inch-diameter rod bent to fit the truss. Members, U_6-L_6, U_6-L_5, U_5-L_5, and U_5-L_4, were also one continuous three-eighth inch-diameter rod. The adjacent one-half inch-diameter members such as U_1-L_1, and U_1-L_2 were continuous.

The construction of the trusses was exactly that which would be encountered on the average construction project. While this provided an opportunity for a very fair load carrying test of the product, it gave rise to a marked difference in the results obtained from the two trusses.

These particular trusses are, normally, spaced two feet, zero inches on centres and thus carry a load per foot equivalent to two square feet of floor. In this case, the trusses were designed for a floor loading of seventy-five pounds per square foot, including the

weight of the floor, or a total load of one hundred and fifty pounds per lineal foot of truss. For convenience, this has been expressed in the results as a load of three hundred pounds per panel point.

GENERAL THEORY

The theory of simply-connected, or pin-connected trusses has been used throughout the calculations.

The theory of buckling, as applied to the top chord of a simply-connected truss, has been adapted from a paper by S. Timoshenko in the 'Proceedings of the American Society of Civil Engineers', 1930, p. 1006. The full calculations for this part of the problem are given in Appendix I.

The nomenclature is tabulated below with the corresponding units, where applicable.

Diagonal-- Sloping web member.

Load-- Action force applied to the trusses, pounds, (lbs.)

Lower Chord-- Members forming the lower perimeter of the truss.

Strain-- Unit strain, micro-inches, (micro-in.).

Total Stress-- Total axial stress, pounds, (lbs.).

Unit Stress-- Stress per unit area, pounds per square inch, (psi).

Upper Chord-- Members forming the upper perimeter of the truss.

Vertical-- Vertical web member

Web Members-- All interior members.

All measurements are expressed in terms of feet and/or inches.

PART II

TEST PROCEDURE

a. Preparation

The first step in the test procedure was the determination of the actual measurements of the trusses. Since the construction of the trusses was quite rough, the measurements were taken to the nearest one-sixteenth of an inch, it being felt that the overall accuracy of the test would not justify any greater accuracy of dimensions.

All measurements were taken relative to the point of intersection of the centre lines of the vertical and diagonal at each joint. A further measurement was taken in order to reference this point of intersection to either the bottom extremity of the top chord or the top surface of the bottom chord, for upper and lower chord joints, respectively.

Each of the trusses was stamped, on the bearing surface adjacent to joint U_0, with a reference number. Thus the trusses are referred to as Truss No.1, or Truss No.2, throughout this report.

After the measuring had been completed a short length of the middle section of each member to which gauges were to be applied was prepared for the application of the gauges.

Before anything further could be done with the trusses it was necessary to set up some firm foundations on which to support them.

The supports which were finally decided upon are shown quite clearly in several of the pictures depicting the loading of the trusses. They consisted of two, approximately three-foot, six-inch lengths of twelve-inch 'I-beam'. These were placed one at each end of the test structure. The flat bearing surfaces of the tops of the beams thus duplicated the conditions encountered in a

field application of these joists. However, the twelve-inch depth was insufficient to provide the necessary clearance between the bottom chords of the trusses and the floor. Hence, it was necessary to build up pedestals to raise the whole structure.

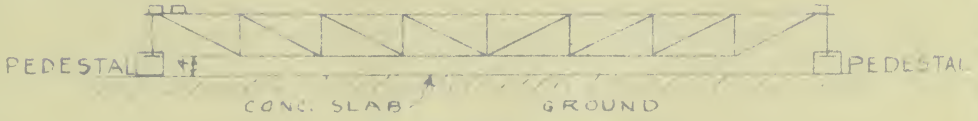
The foundation is depicted, schematically, in Figure 5. Each steel 'I-beam' was set with the ends resting on boards containing four bolts arranged in such a manner that the heads of the bolts rested on the floor while the nuts on the bolts were in contact with the underside of the board. Thus the ends of the beam could be raised, lowered or tilted in the same manner as the head of an engineer's transit or level.

At the same time wooden forms were attached to the bottom flange of the beam, directly below the points on which the trusses were to bear. These forms were for the purpose of pouring the supporting pedestals and were, approximately, eight inches square.

When the foundations had been readied in this manner the beams were aligned and levelled by the use of a transit and an engineer's level. The pedestal forms were then sealed with Plaster of Paris and filled with a mixture of sulphur and fire clay. This was obtained from the cylinder capping equipment in the concrete laboratory. Cubes of this material had previously been tested in compression to ensure that it would have sufficient bearing strength for the purpose and, as an extra precaution, the wooden forms were left around the outside.

The main reasons for using this particular material were its rapid setting action and the fact that it was readily available in the laboratory. It had one serious disadvantage, namely, its shrinkage properties. Upon setting, all free surfaces undergo a notable recession. Thus it was necessary to maintain a high liquid level when pouring in

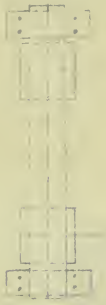
PLAN



ELEVATION

TRUSS ARRANGEMENT

FIGURE 4



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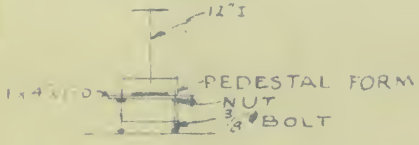


ELEVATION

FOUNDATION LEVELLING STANDS

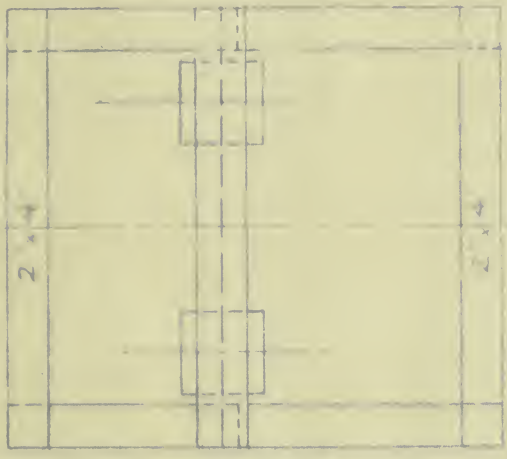
FIGURE 5

order that there would be no shrinking away from the underside of the beams. (It should be noted that, when the equipment was dismantled, all bearing surfaces were found to be smooth and firm and the main body of the pedestals was perfectly sound.)

As soon as the pedestals had hardened the levelling boards were removed and a four-foot length of dressed 4"x4" was placed under each end of the 'I-beam' between the pedestal and the outside edge as shown in Figures 6 and 7. These were, in turn, joined by lengths of 2"x4" thus making a rectangular frame.

The trusses were then placed on the beams exactly two feet apart and three sets of cross braces were welded to them at approximately the quarter and mid points of the trusses. After re-checking the alignment and spacing the ends of the trusses were spot-welded in position. A sketch of the arrangement is shown in Figure 4. Then, in order to prevent overturning of the end supports, lengths of 2"x4" were placed from the wooden frame to the top of the web and the lower surface of the upper flange on the inside of the structure. Similar braces were placed from the frame to the bottom of the web on the outside. This may also be seen in Figures 6 and 7.

View of Structure from Southeast
Fig. 6



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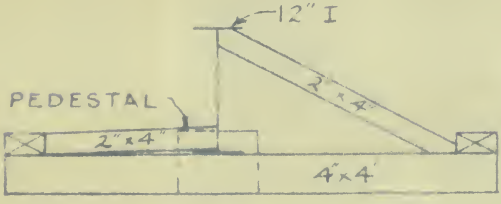


ELEVATION

FOUNDATION

FIGURE 7

Next, the gauges were applied to the members previously prepared using the method described in Appendix II. The Gauge Data is tabulated in Table 2 of Appendix I.

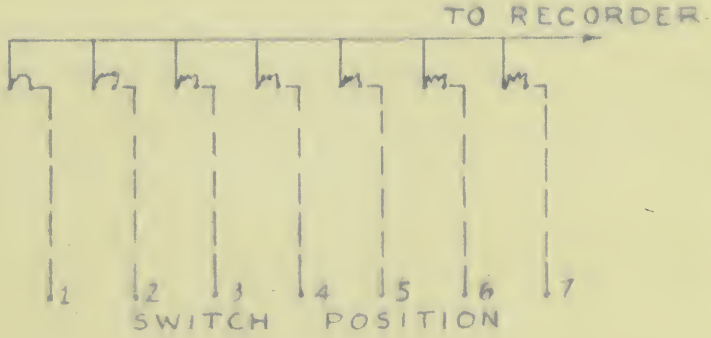
Only four of the gauges were lost. Three of these had insufficient leakage resistance and one gauge lost most of its internal resistance.

At this stage two temperature gauges were attached to a three-quarter-inch diameter bar of structural steel. One gauge was of the 'A-1' type, while the other was of the 'A-3' type. After the lead wires had been attached to these two compensating gauges the bar and leads were firmly taped to a length of 2"x2" lumber to prevent damage to the gauges and this whole assembly was placed on the floor directly below the trusses near the western support. This may be seen, although not too clearly, in the bottom of Figure 8. Figure 9 shows the location of the remaining gauges.

Truss No. 2--Member U_3-L_3
Fig. 8

MEMBERS HAVING GAUGES

FIGURE 9



ILLUSTRATIVE CIRCUIT

FIGURE 10

All the gauges were connected to a single common lead, making certain that the length of lead between any one gauge and the recorder terminal would be the same as the length of the compensating gauge leads. Then sixty-eight single leads were cut, each of the same length. This length was such that the resistance from the recorder terminal, through the switch box, to the gauge would be the same as the resistance of one of the compensating gauge leads. The latter were approximately ten feet in length and all leads were of number twenty, rubber coated, solid, single lead-in wire.

One lead was connected to each gauge and each of these leads was connected to a separate terminal on a switch box. This, of course, meant that sixty-eight switch positions must be provided and switch boxes had to be prepared.

Two switch boxes were used. The first of these contained one twelve position, single circuit, single throw, selecter switch and one eleven position, double circuit, single throw, selecter switch. The twelve position switch had an 'off' position as well, thus allowing full utilization of the twelve positions. The second switch, however, had no 'off' position and, hence, was limited to ten positions on each circuit, or twenty positions in all. These switches were designated 1 and 2, respectively, and the circuits on switch 2 were indicated by the notation 2-1, or 2-2, thus giving rise to switch positions such as 1-1, 1-2, 2-1-1, 2-2-1, etc.

The second box contained three six position, single circuit, double throw switches, designated A, B and C. Since these switches had 'off' positions it was possible, by connecting each side of a switch to separate terminals, to obtain twelve positions on each switch. Essentially, this amounted to converting them to two circuit, single

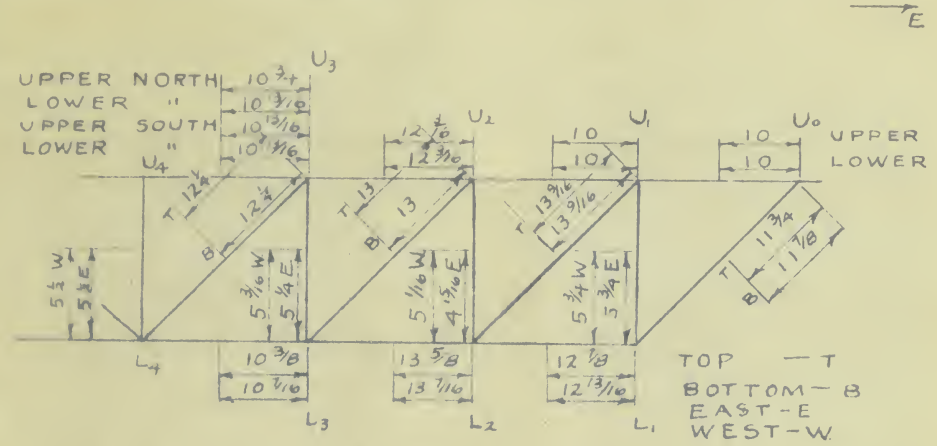
throw switches. These circuits were designated A1, A2, B1, B2, C1 and C2, giving position designations such as A1-1, A1-2, C2-1, etc. Figure 10 shows an illustrative circuit diagram.

The common lead from all the measuring gauges was connected to one terminal of the recording box and a lead from the other terminal was connected to a circuit terminal on the switch box. It was, of course, necessary to have a separate circuit terminal for each switch circuit or a total of nine. Three of these were on the two-switch box and the other six were on the three-switch box. Since the readings were taken so that all 'A-1' type gauges were read first and then all 'A-3' type, it was only necessary to change the compensating gauge leads once during each set of readings.

The recorder and switch boxes were set upon a table in front of the structure as shown in Figure 11.

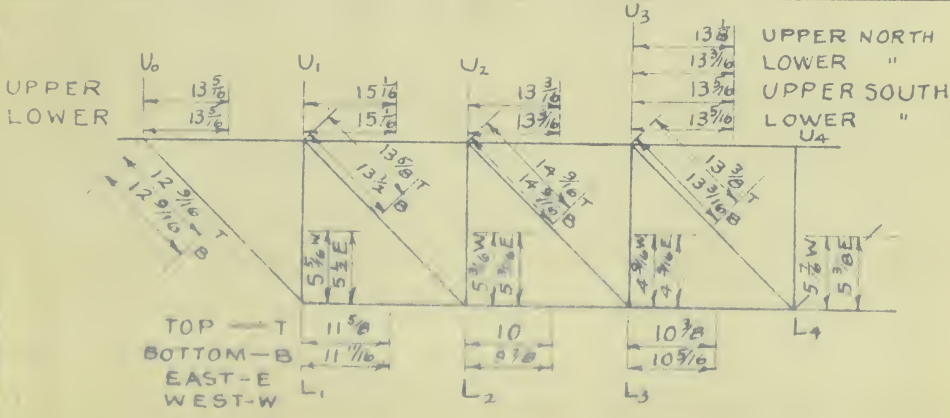
Recording Station
Fig. 11

With everything else in readiness, the positions of the gauges, relative to the adjacent panel points, were measured and recorded. This information is given in Figures 12 and 13.



GAUGE POSITIONS - TRUSS NO.1

FIGURE 12



GAUGE POSITIONS - TRUSS NO.2

FIGURE 13

The leads from the gauges were positioned to prevent a short circuit due to outside contact and securely fastened, just below the gauge, to prevent excessive movement. The tape fastenings and the position of the leads are well shown in Figure 8 and Figures 14 to 20, inclusive.

Deflection gauges were arranged by setting a steel scale on a block of wood which had been secured to the mid point of the lower chord, and holding it upright by means of a sliding bracket as shown in Figure 20. A fine thread was then attached to the 'I-beam' support at each end and drawn taught. This was used as the datum since it would eliminate the settlement of the supports. As an added measure an Ames dial was set on the upper surface of the lower chord at the same point on truss number one. This may also be seen in Figure 20.

b. Loading

After a set of zero readings had been obtained a loading deck was mounted on the structure in the form of two foot, six inch lengths of 2"x4", spaced approximately six inches on centres. Spikes were placed through drilled holes on each side of the upper chords in order to provide lateral support between trusses. The underside of this decking shows very clearly in Figure 15.

When the decking had been applied with its attendant pounding and the application of various weights of men and materials it was found that an appreciable change had been occasioned in the zero readings. It was felt that sufficient preliminary load had been applied to take the slack out of the gauges. Hence, loading was commenced.

The weight of forty bricks was taken as the basic unit for the determination of loading intensity. Two such determinations were

Truss No.2--Member U_1-L_1
Fig. 14

Truss No.2--Member U_4-L_4
Fig. 15

Truss No.2--Member U_2-L_2
Fig. 16

Truss No.1--Member U_2-L_2
Fig. 17

Truss No.1--Member U_3-L_3
Fig. 18

Truss No.1--Member U_1-L_1
Fig. 19

Truss No.1--Member U_4-L_4
Fig. 20

made, one at the beginning and one at the end of the test. This data is recorded in Table 3 of Appendix I.

In loading the bricks were spaced one-half inch apart longitudinally and as close as possible at right angles to the centre line of the structure. The first five courses were laid as stretcher courses and then alternate rows, starting with row 6, were made header courses. The objects of this arrangement were to prevent arching action in the longitudinal direction and to obtain as much arching effect as possible in the normal direction. This would help to ensure a uniform distribution of loading longitudinally and, at the same time, the arching action normally would tend to equalize the loading carried to each truss. The latter would also tend to minimize any side-thrust effect due to rotation of the ends of the 2"x4"spacers in the platform arising from deflection of the platform members. Figures 6, 21 and 22 show the loading of the trusses quite clearly.

Fig. 21

View from Southwest during Loading

View from Southeast--Final Load
Fig. 22

Two courses of brick were applied to the trusses and the structure was allowed to stand until conditions had reached equilibrium. Then a set of readings were taken. From then on, the structure was loaded one course at a time and then allowed to reach equilibrium before readings were taken. After each set of readings the data obtained from the first few gauges was re-checked to see that the system had actually been in equilibrium. In the case of the hundred and two pound panel point load, it was found that conditions had altered during the time of reading and a new set of observations had to be made. This was the only case of its kind during the experiment. The data is included in Tables 4 to 33 of Appendix I.

During the test trouble was encountered in the maintenance of the datum cords for the deflection gauges. Since it was impossible to get in behind the structure, deflection readings for Truss No.2 were abandoned. There had been no indication of deflection in the floor at this time hence further deflection observations were made, using the floor as a datum, on Truss No.1. The deflection data is summarized in Table 34 of Appendix I.

Upon completion of the test members, U_0-L_1, U_3- L_4, and L_3- L_4, were removed from Truss No.1 and subjected to a tension test to determine the modulus of elasticity. The values obtained checked fairly well with the value of 29.0×10^6 p.s.i. which was assumed. The data is given in Tables 35 to 37 and Figures 1 to 3 of Appendix I.

PART III

RESULTS

1. Total Stresses

The results of the testing were first recorded in terms of the total stress in each member and this value was compared to the theoretical value, based on analysis by the simply-connected theory and the actual dimensions of the trusses. In Appendix I the observed total stresses are given in Tables 4 to 33 and the computed total stresses are given in Tables 38 and 39. Samples of all calculations involved are given in the same Appendix.

Figures 23 to 37, inclusive, depict the total stress results graphically.

With reference to Figure 23, it will be seen that the theoretical and observed values differ very markedly. These graphs are for the members U_0-U_1 of each truss and, since these are the end members adjacent to the bearing surface, it is difficult to ascertain exactly what caused the discrepancy. In the first place, a horizontal component could be transmitted from the end to the support since the supports were anything but frictionless. Secondly, it would appear, at least in the case of Truss No.1, that an error had been made in the initial reading. Since the test was carried to failure, no check could be made on these initial values. In view of these uncertainties, the horizontal component of the observed stress in the member, U_0-L_1, has been plotted for each truss. This value should correspond to the stress in U_0-U_1 and agrees fairly well with the theoretical values.

It is interesting to note the resemblance of the graph in

Figure 23 to that of Preliminary Test No. 1, shown in Appendix III, where no initial loading was applied before readings were started. It is possible that, U_0-U_1 being a stiff member with a comparatively low stress, the initial loading was not sufficient to overcome the slack in the gauges.

The remaining figures show a rather good check between the theoretical and observed stresses when it is borne in mind that the locked-in stresses at the welded joints, as well as other secondary stress effects which were indeterminate, would have some bearing on the results.

It will be noted that the observed stresses in the chord members were, if anything, slightly low. This seems quite reasonable when one notes that the values for the web members are also low. The difference is not sufficient to cause concern. The greatest discrepancy appears to be in the upper chord members which may well have been affected throughout by the conditions at the supports.

In practically all cases the observed results from Truss No. 2 are lower than those for Truss No.1. This probably indicates that a slightly larger proportion of the load was carried by No.1.

In the case of the web members the observed results are somewhat lower than the theoretical. This is indubitably due, at least in part, to an indeterminate amount of the shear being absorbed by the chords.

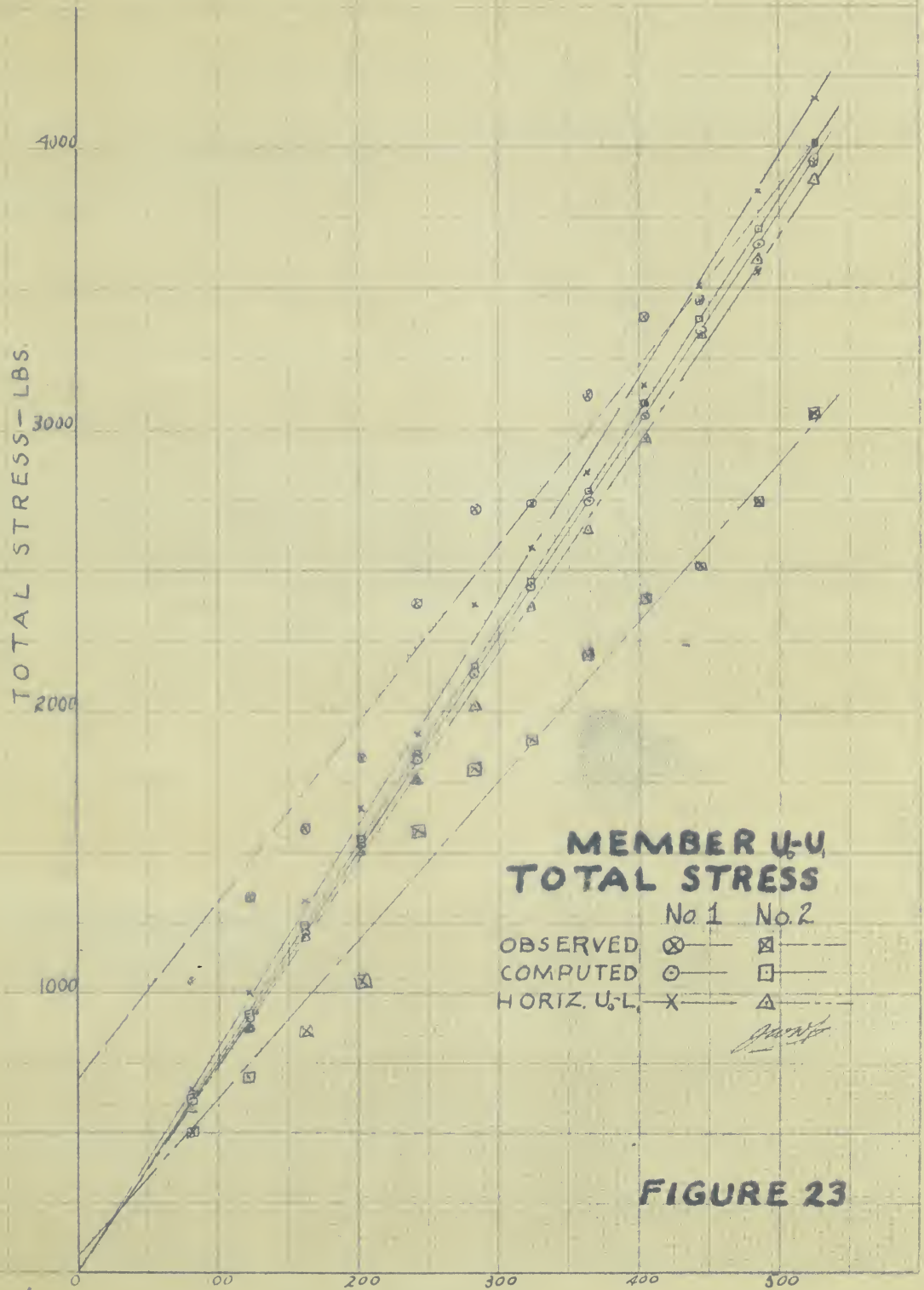

TOTAL STRESS - LBS.
4000
3000
2000
1000
0
100
200
300
400
500
LOAD - LBS. / PANEL PT.
MEMBER U₀U₁
TOTAL STRESS
No. 1
No. 2
OBSERVED
COMPUTED
HORIZ. U₀L₁

FIGURE 23

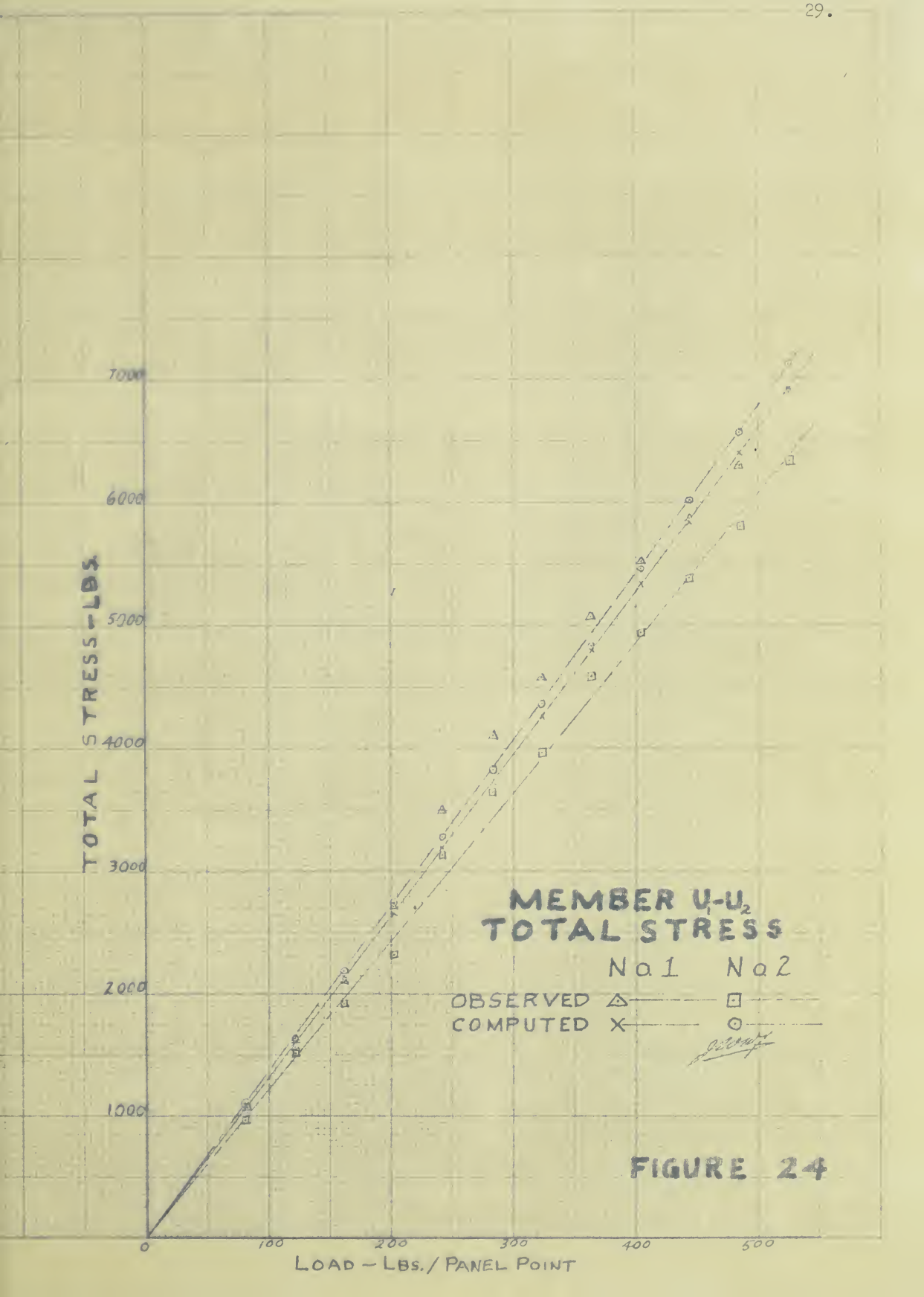

TOTAL STRESS - LBS.
7000
6000
5000
4000
3000
2000
1000
0
100
200
300
400
500
LOAD - LBS. / PANEL POINT
MEMBER U1-U2
TOTAL STRESS
No 1 No 2
OBSERVED
COMPUTED

FIGURE 24

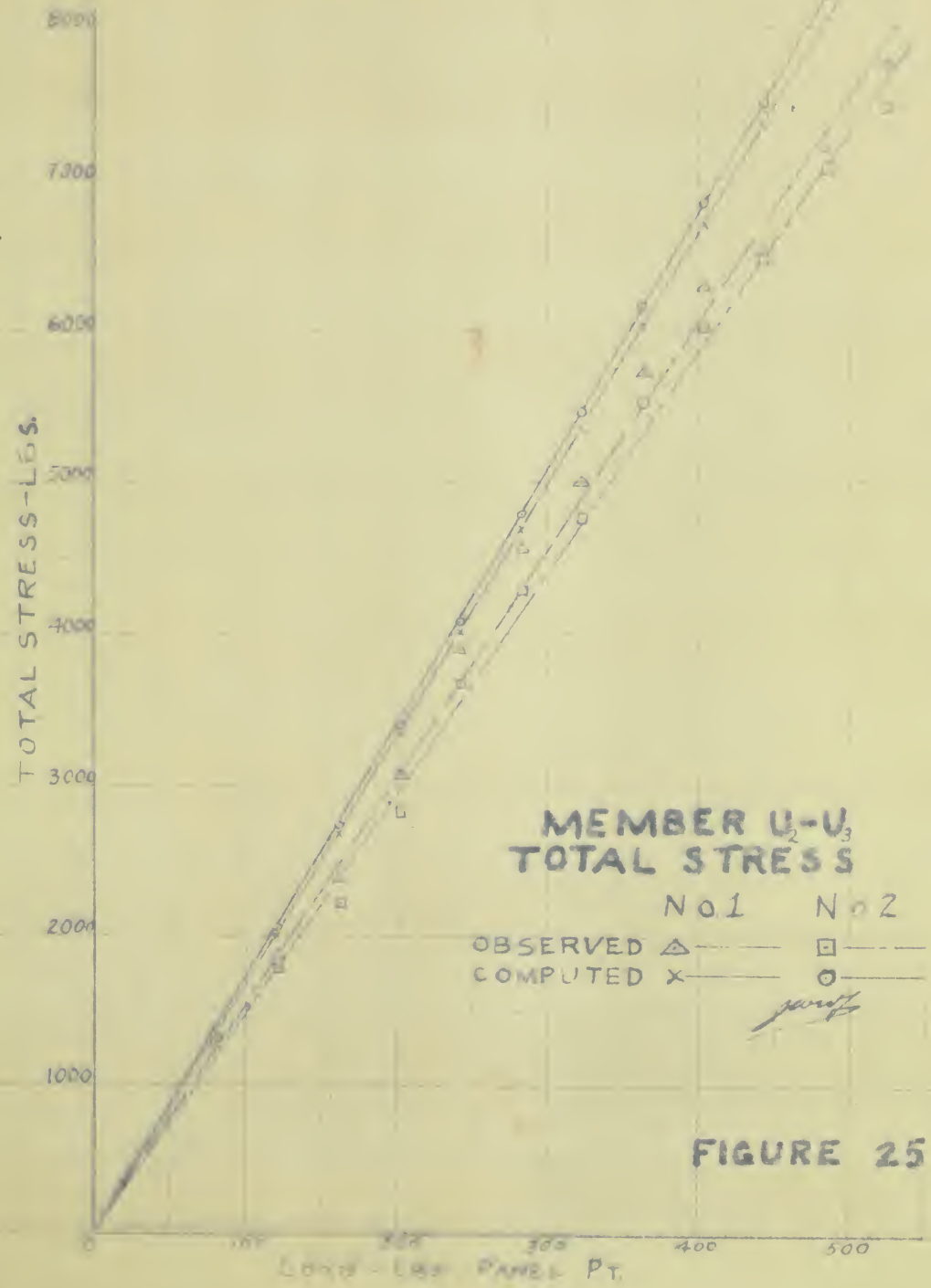

8000
7000
6000
5000
4000
3000
2000
1000
0
TOTAL STRESS-LBS.
100
200
300
400
500
LOAD - LBS. PANEL PT.
MEMBER U2-U3
TOTAL STRESS
No 1
No 2
OBSERVED
COMPUTED

FIGURE 25

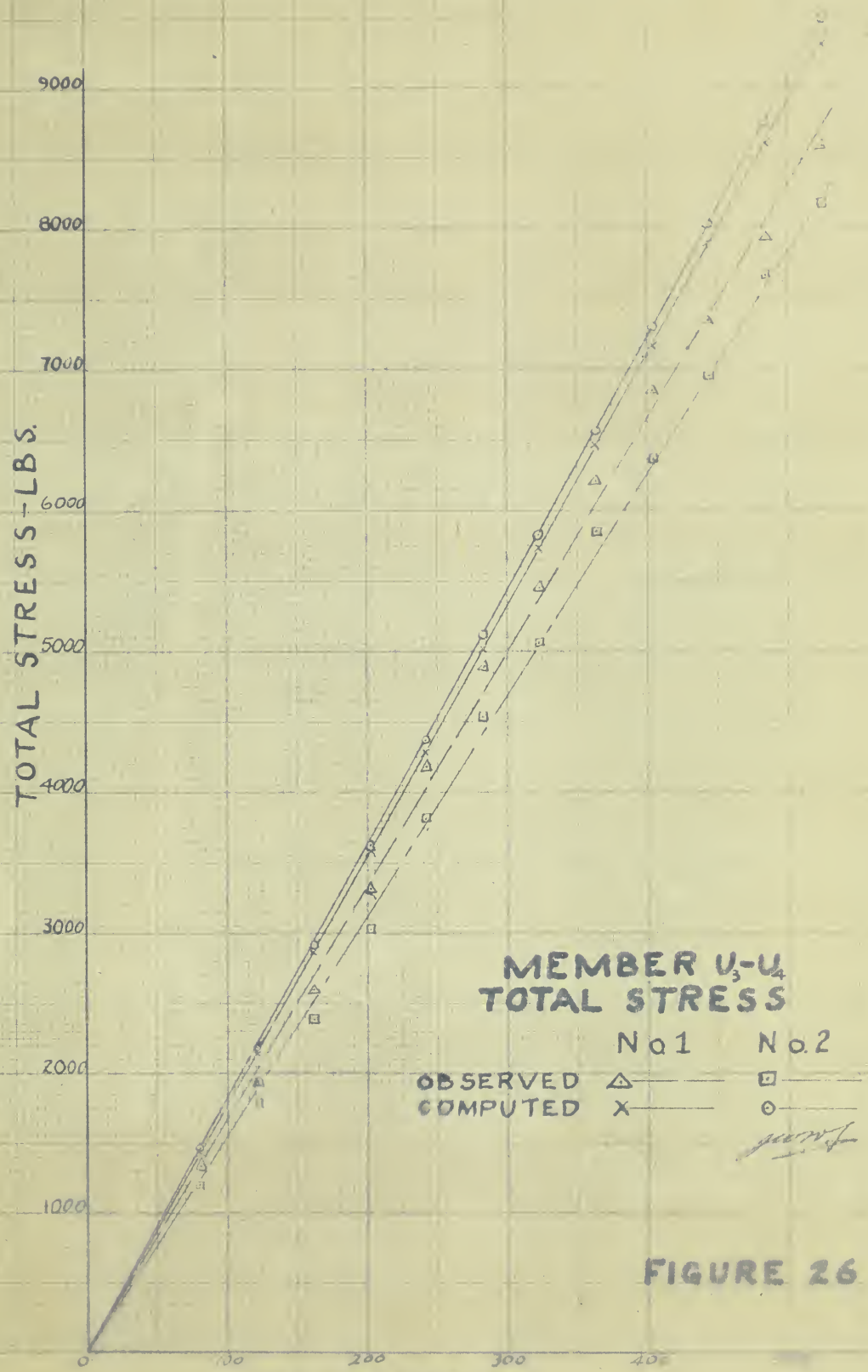

TOTAL STRESS-LBS.
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
100
200
300
400
LOAD-LBS./PANEL PT.
MEMBER U3-U4
TOTAL STRESS
No 1
No 2
OBSERVED
COMPUTED

FIGURE 26

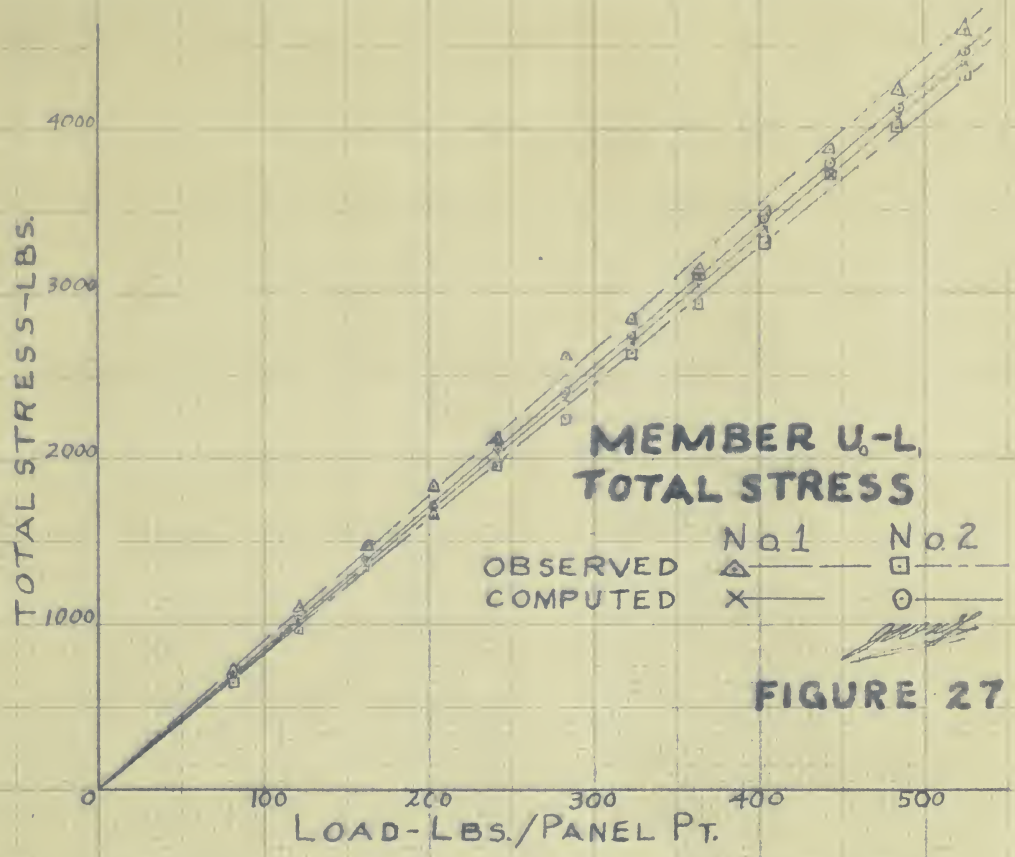


FIGURE 27

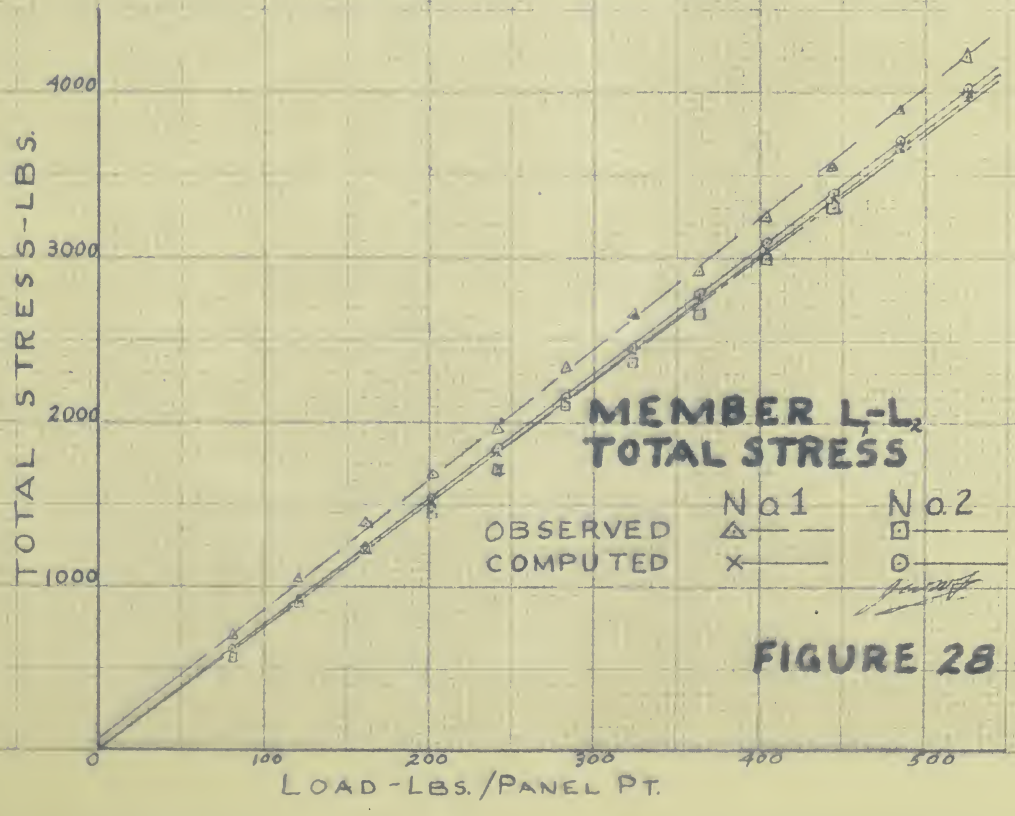


FIGURE 28

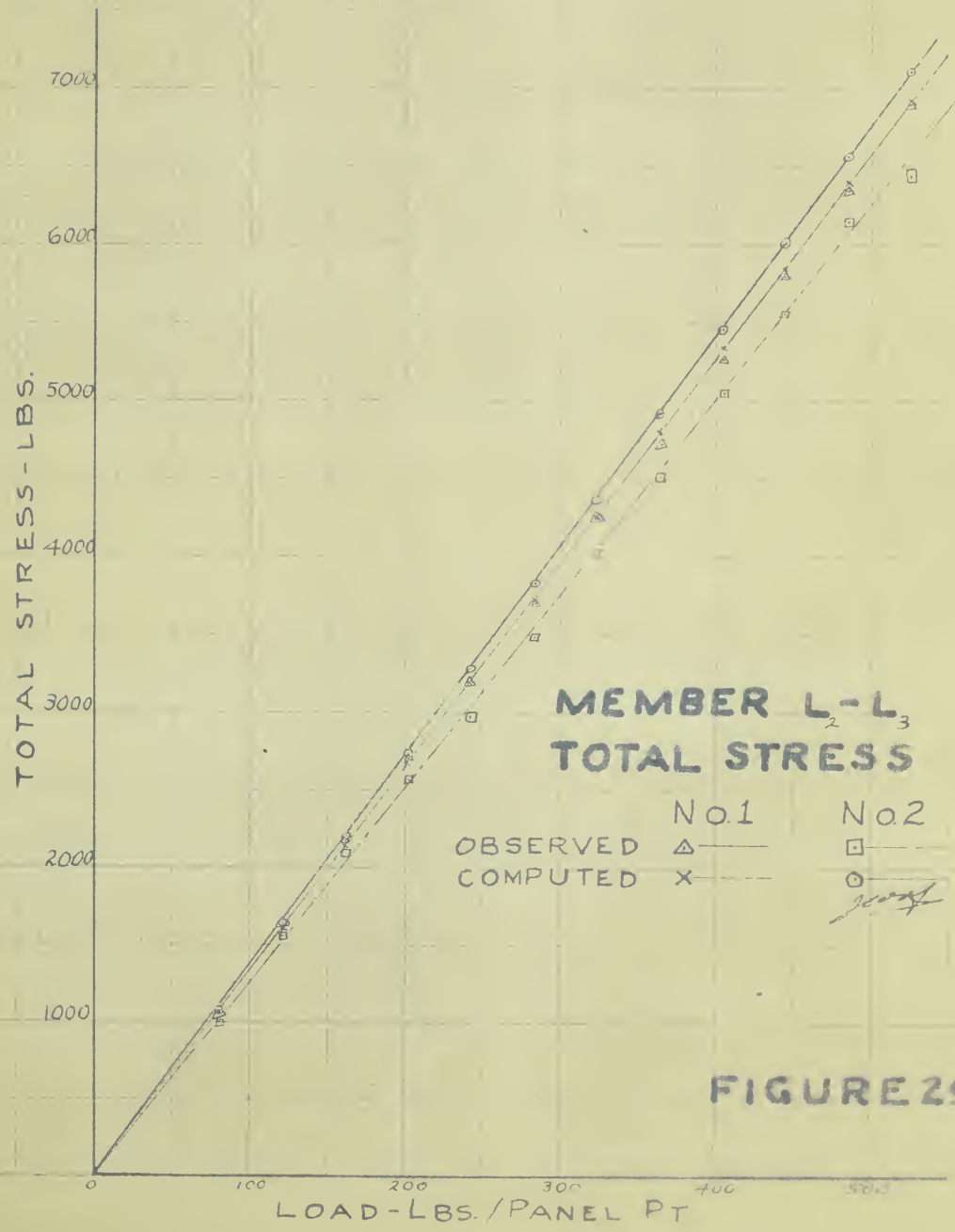

7000
6000
5000
4000
3000
2000
1000
TOTAL STRESS - LBS.
0
100
200
300
400
LOAD - LBS. / PANEL PT
MEMBER L2-L3
TOTAL STRESS
No.1
No.2
OBSERVED
COMPUTED

FIGURE 29

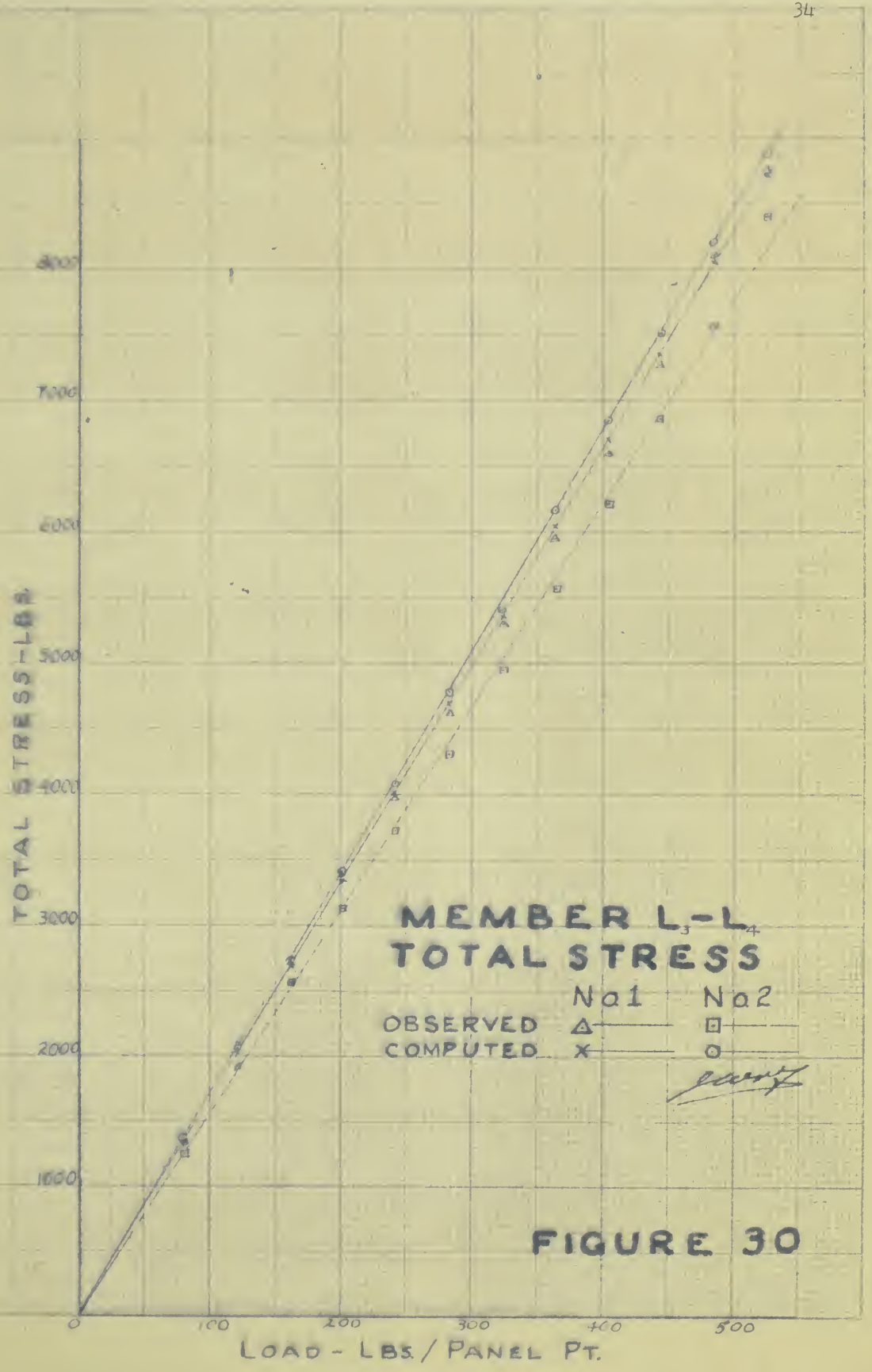

MEMBER L3-L4
TOTAL STRESS
No1 No2
OBSERVED △ □
COMPUTED X O
TOTAL STRESS-LBS.
8000
7000
6000
5000
4000
3000
2000
1000
0
100
200
300
400
500
LOAD - LBS / PANEL PT.

FIGURE 30

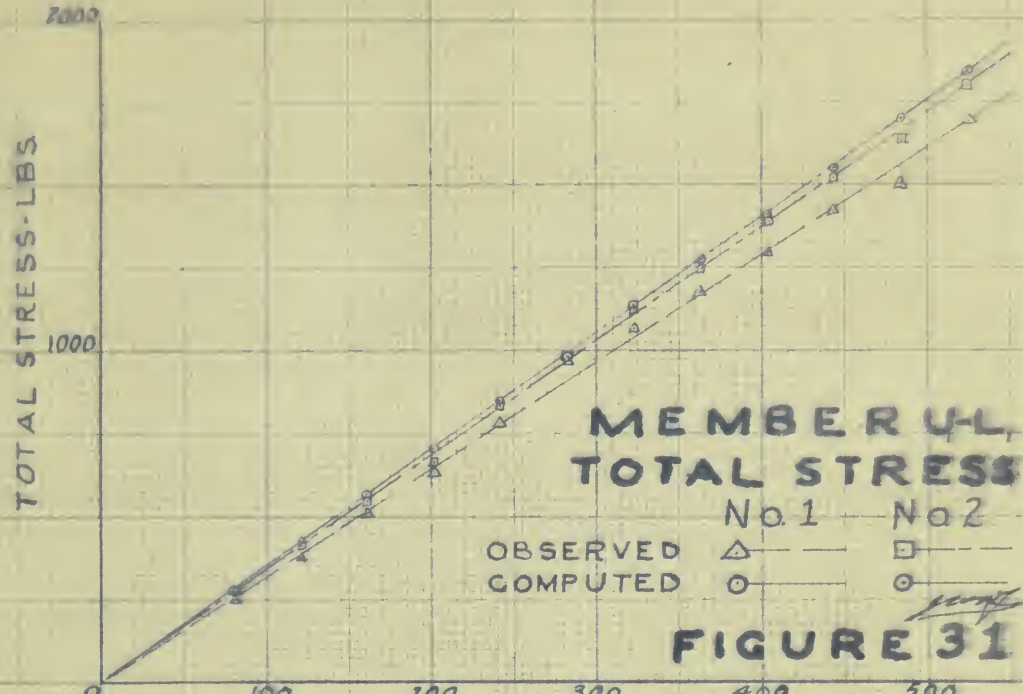


FIGURE 31

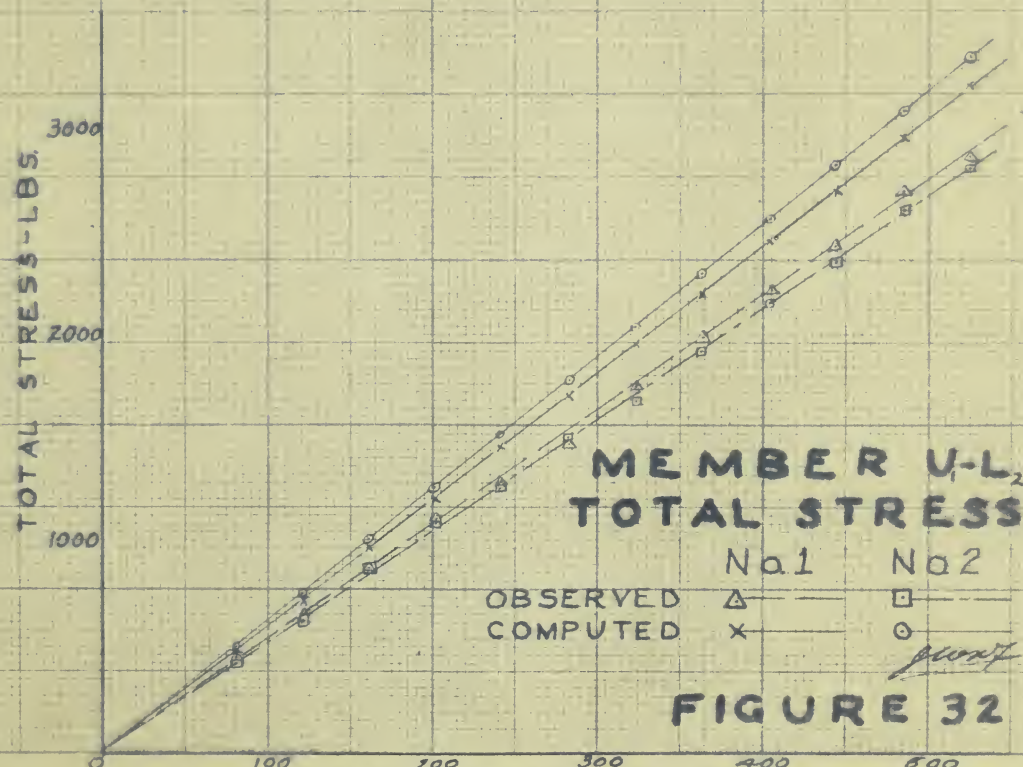


FIGURE 32

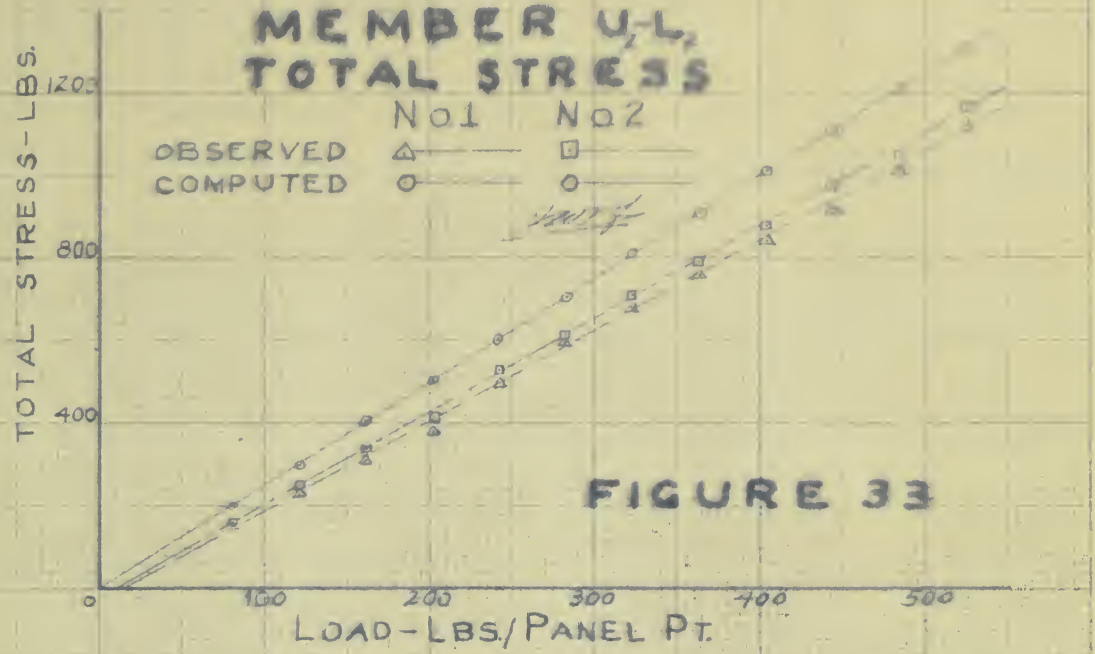


FIGURE 33

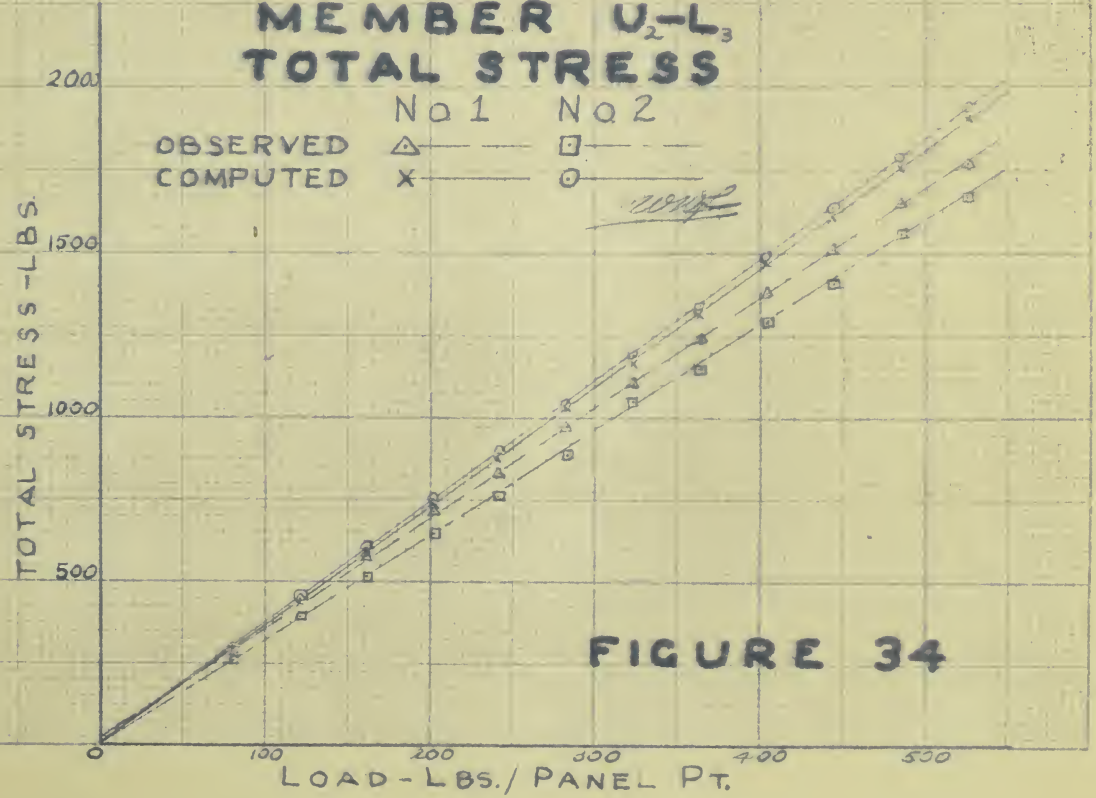


FIGURE 34

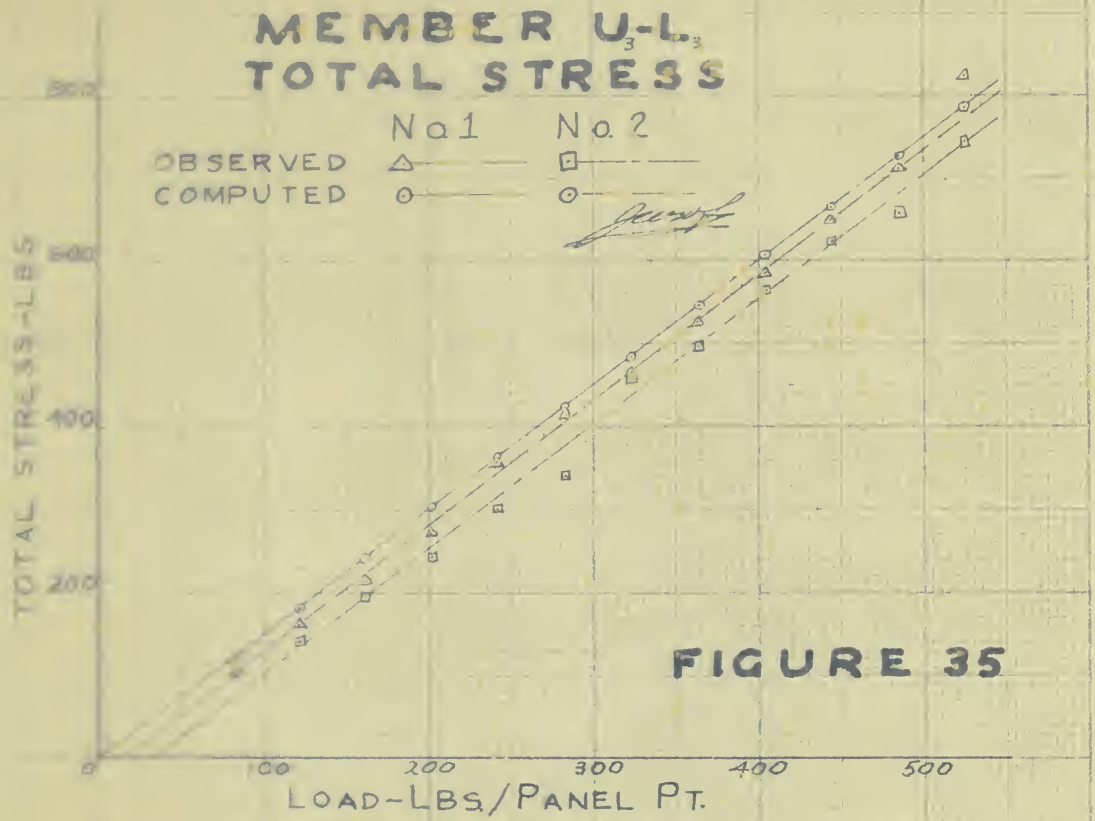


FIGURE 35

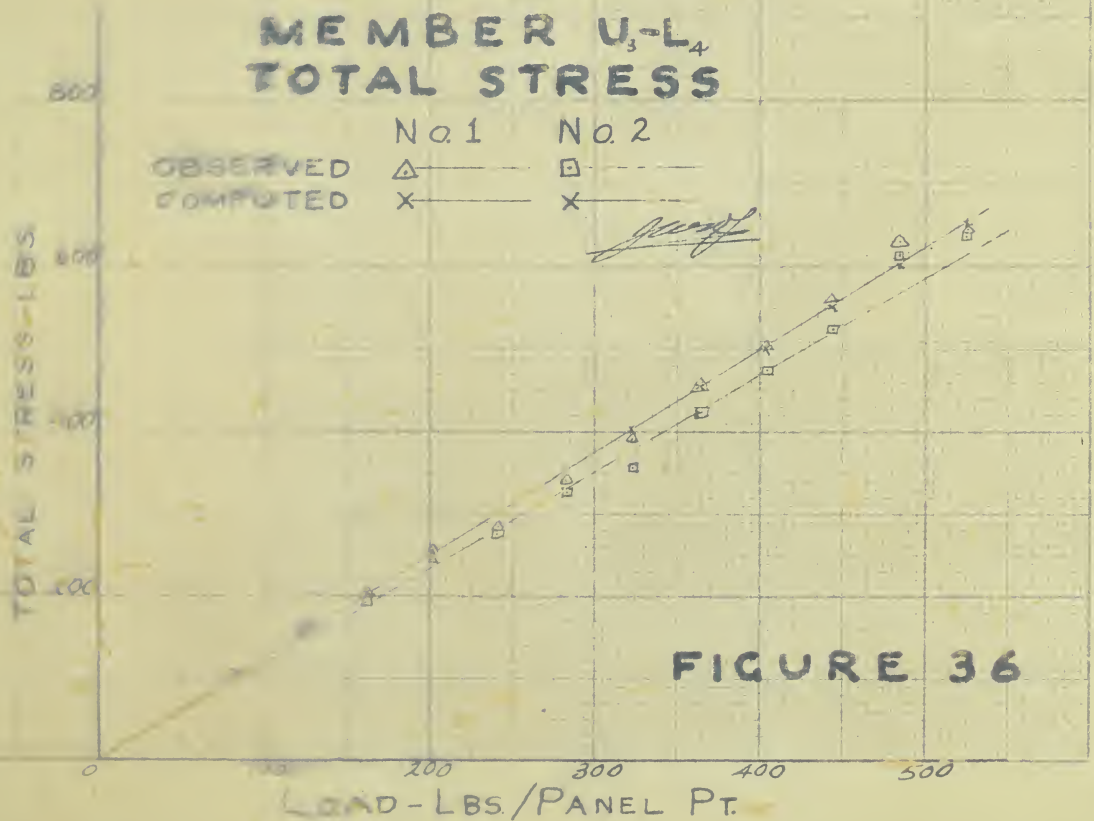


FIGURE 36

MEMBER U_4-L_4
TOTAL STRESS

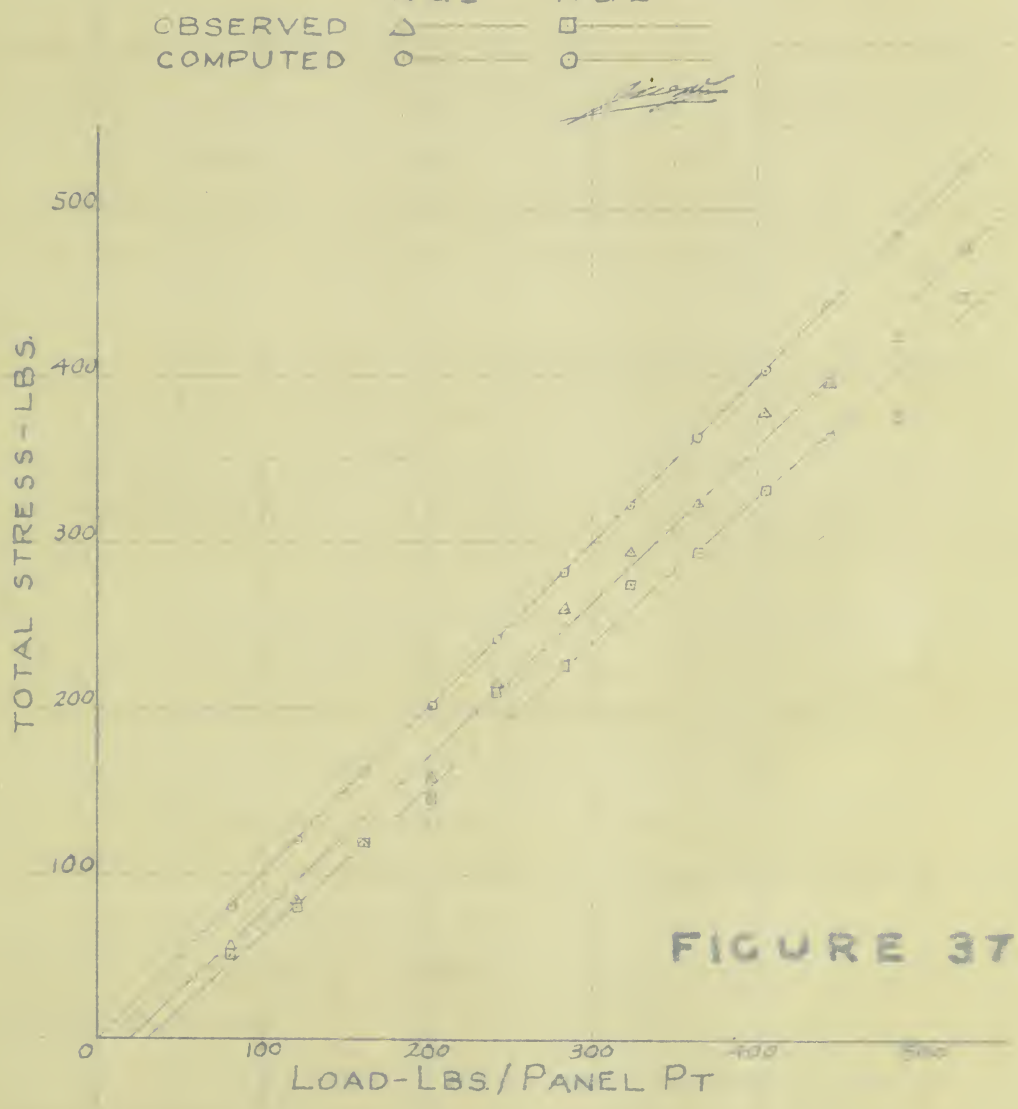


FIGURE 37

2. Maximum Unit Stresses

In the second analysis, the maximum unit stresses encountered in each member, including bending effects, were compared to the theoretical maximum based on the pin-connected theory and axial loading except in the case of upper chord members as will be mentioned later. The computed data for this analysis is given in Tables 40 to 43 of Appendix I and samples of the attendant calculations are given in the same section. The graphical presentation is made in Figures 38 to 52, inclusive.

In the case of all of the upper chord members, the computed values are based on the assumption of axial load plus bending stress due to a uniformly distributed load between panel points. A study of the related calculations in Appendix I will indicate the assumptions made.

In the case of the members, U_0-U_1, shown in Figure 38, there is no correspondence of worthwhile merit between the observed and the computed values. In view of the discrepancies shown in Figure 23, these particular results are of little value.

The remaining results are of some interest. Since it is difficult, if not impossible, to tell just what the curve should be in these cases, the curves have been drawn to fit all the points possible with the result that some of the graphs present a rather strange appearance.

Most of these curves exhibit properties which are characteristic of so-called "locked-in" stresses as would be expected in a structure of this nature. In addition, Figures 39, 40 and 41 show, very definitely, that the maximum observed unit stresses were well

below the theoretical values. This can probably be attributed to the method of connection of the upper chord joints. As mentioned in the description of the trusses, this was such that the secondary bending moments would tend to counteract those due to the distributed loading along the chord. Oddly enough, the maximum unit stress appears to have occurred in member U_2-U_3, instead of U_3-U_4, although this might be due to some extra support contributed to U_3-U_4 by the cross braces as well as to the positions of the gauges along the members.

The discrepancy between the computed and actual values in these figures is of the same relative magnitude for both trusses.

Turning to Figures 42 to 45, we find a slightly different picture. Firstly, in all cases, the variation between actual and computed values for the two trusses are of vastly different magnitudes, the greatest discrepancy arising in each case in Truss No.1. The only logical cause to which this may be attributed is the variation in the construction of the two joists.

In two cases, namely members, L_2-L_3 and L_3-L_4, the results for Truss No.2 show a fairly close correspondence to the computed values. The remaining lower chord results show very marked differences.

An examination of the decidedly eccentric connection used at the joint, L_1, as shown in Figures 14 and 19 shows the main reason for the very large secondary bending stresses introduced into members, L_0-L_1, and L_1-L_2. In this case, the additional stress was not detrimental to these tension members, since the same section was used throughout the lower chord and the maximum stress in each case occurred in the member, L_3-L_4. However, as will be pointed out later, this eccentricity may very well have affected the performance of the trusses.

The last two observed values of the stress in L_3-L_4, for both trusses, are probably beyond the yield point of the steel and are thus meaningless. However, this graph clearly indicates an appreciable increase in the stresses due to the secondary effects. In the case of Truss No.1 this is in the neighborhood of twenty-two per cent of the computed stress at the design load.

In the case of the web members one of the most interesting points is the discrepancy apparent in the case of member U_1-L_1. This again is, no doubt, at least partly due to the eccentricity of joint L_1. In the case of Truss No.1 this discrepancy has the astounding value, at design load, of approximately one hundred and sixty-seven per cent of the computed stress.

Again, there is little correspondence between the results obtained from the two trusses. An examination of Figure 48 shows a wide divergence between the actual and computed results for Truss No.2, while the observed results from Truss No.1 follow the theoretical plot very closely. In Figure 49 the reverse is true although to a lesser extent.

The diagonals, as shown in Figures 50, 51 and 52, show a closer agreement with the theory. However, the results for Truss No.1 in Figure 52 show a marked discrepancy with an error, at the design load, of approximately thirty-two per cent of the computed stress.

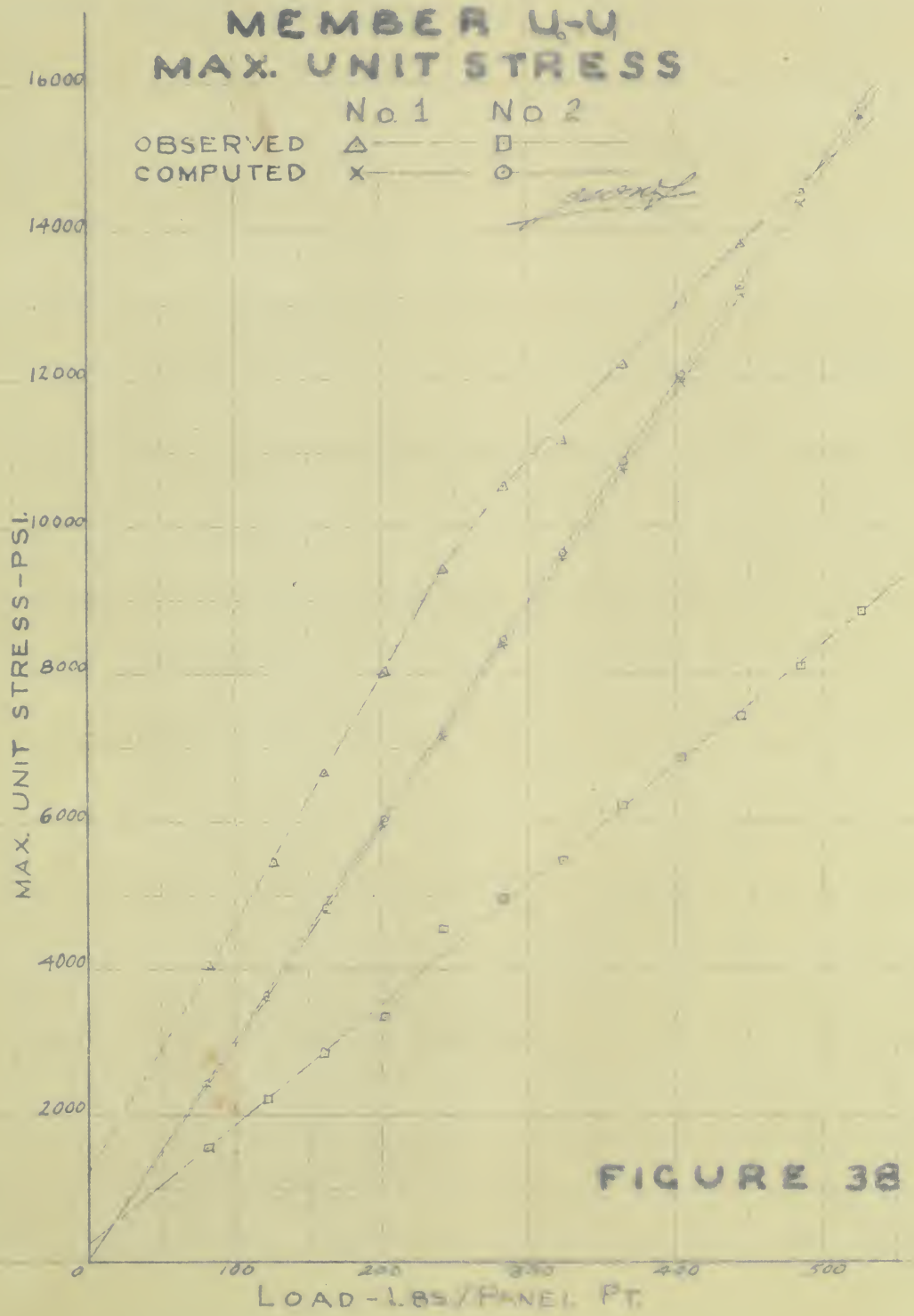

MEMBER U6-U7
MAX. UNIT STRESS
No. 1 No. 2
OBSERVED
COMPUTED
16000
14000
12000
10000
8000
6000
4000
2000
0
MAX. UNIT STRESS-PSI.
100
200
300
400
500
LOAD-LBS/PANEL PT.

FIGURE 38

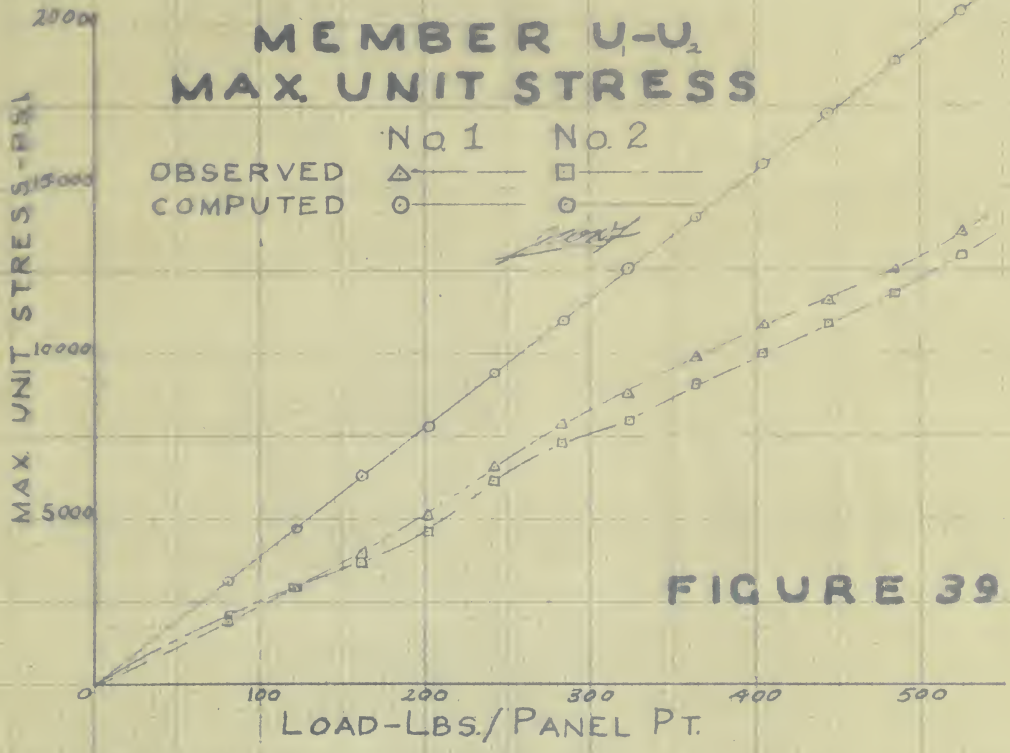


FIGURE 39

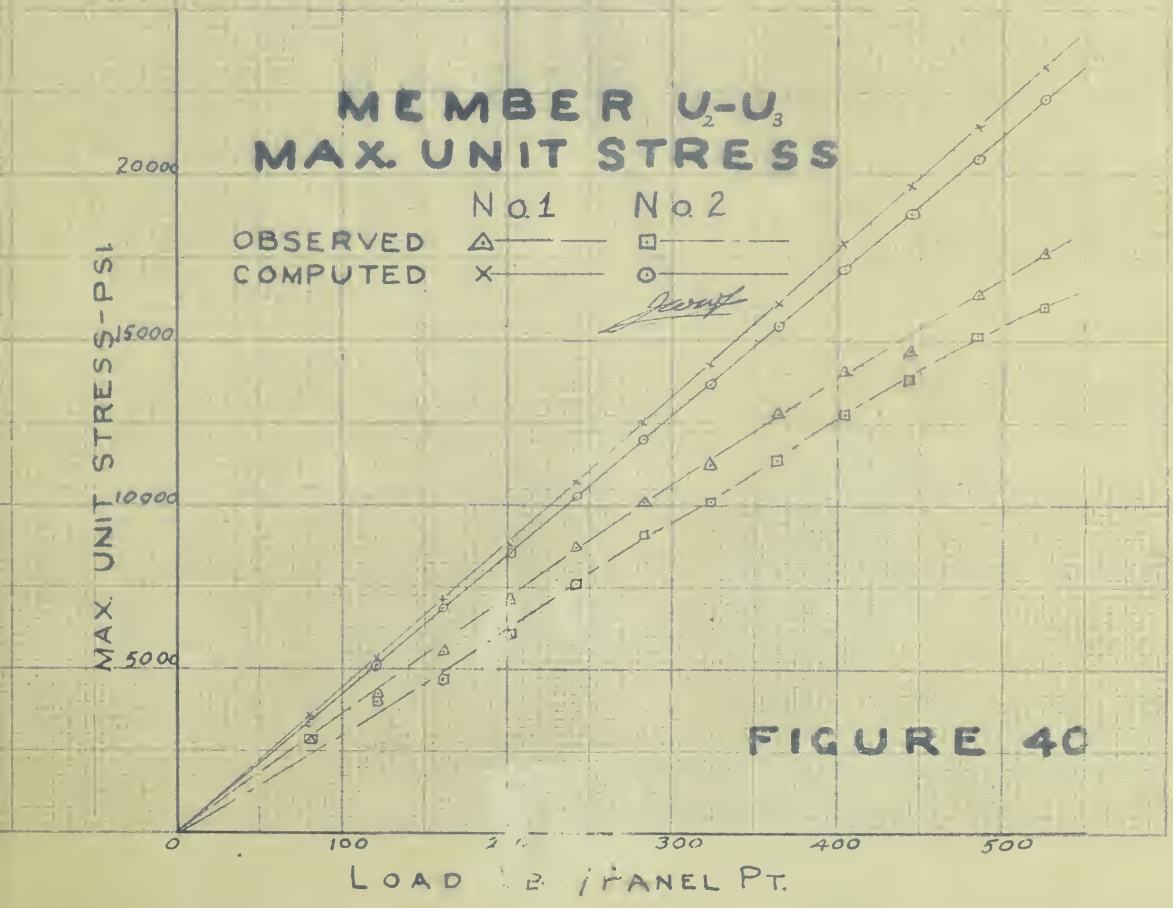


FIGURE 40

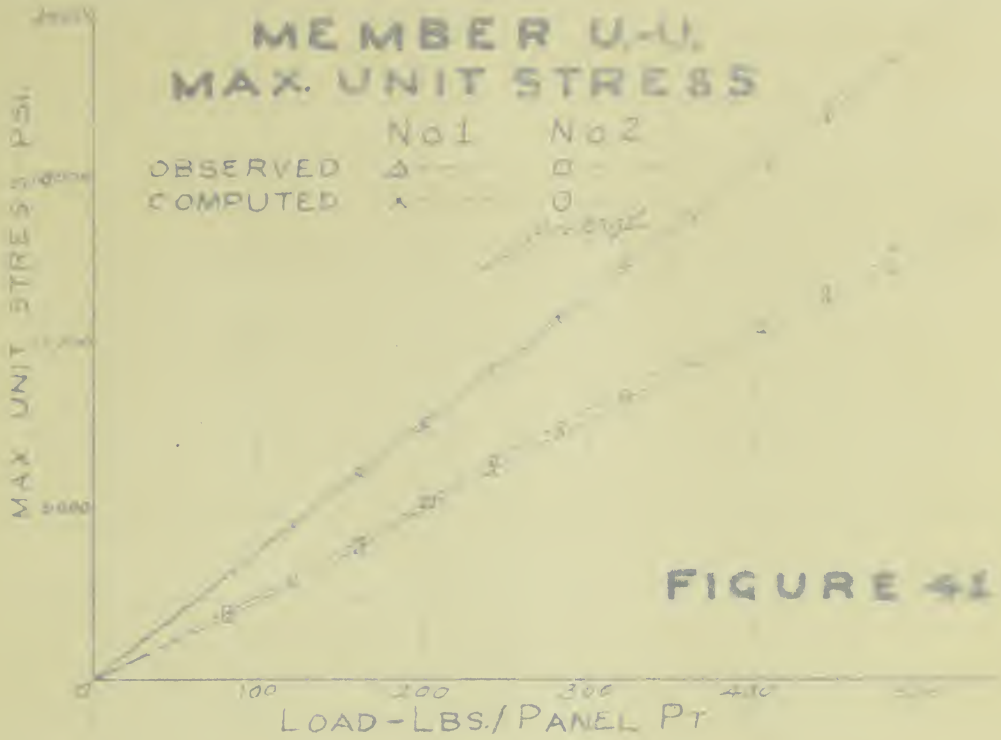


FIGURE 41

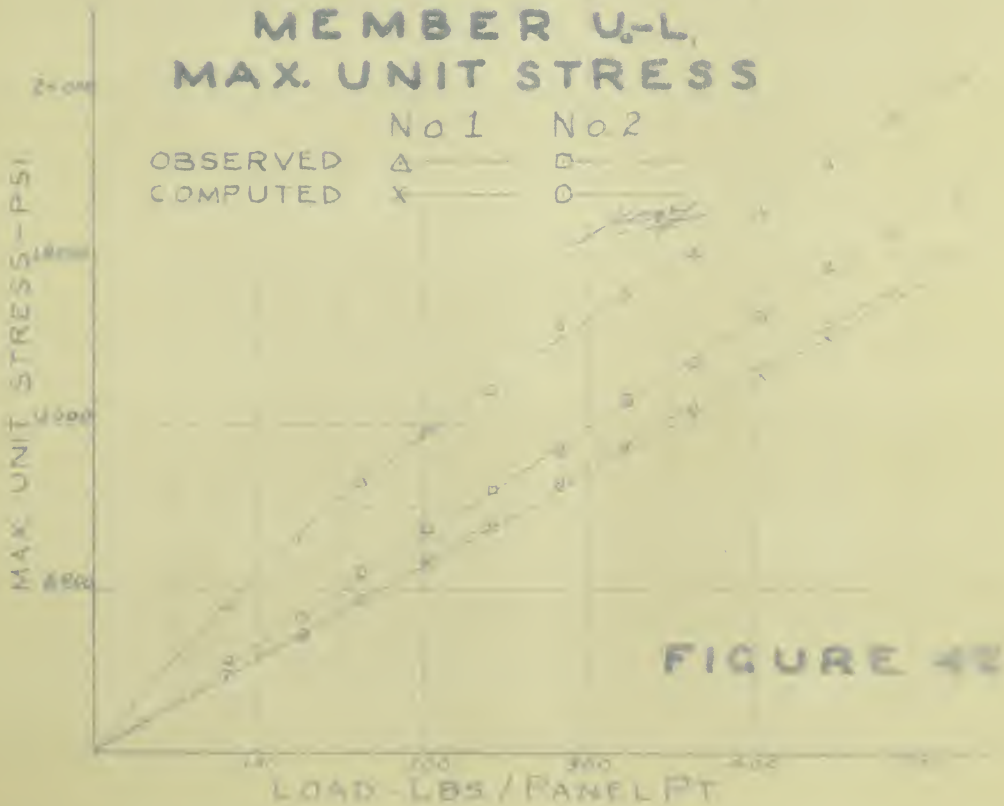


FIGURE 42

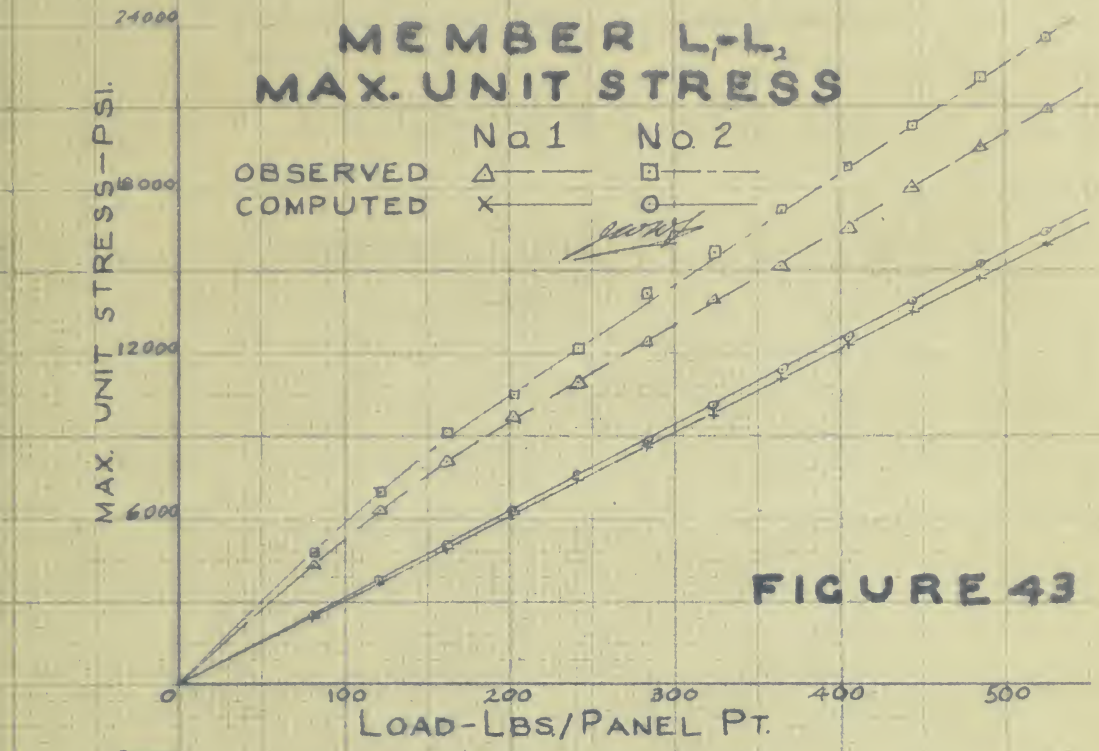


FIGURE 43

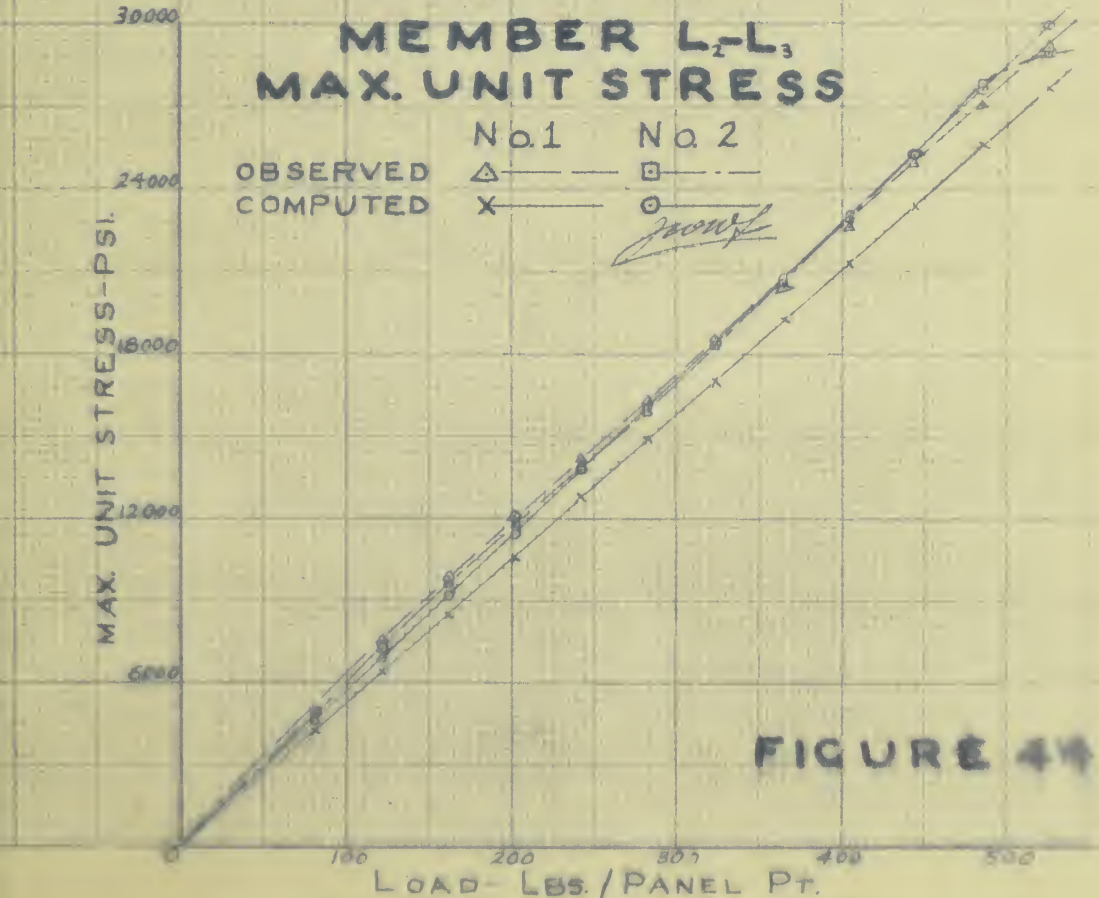


FIGURE 44

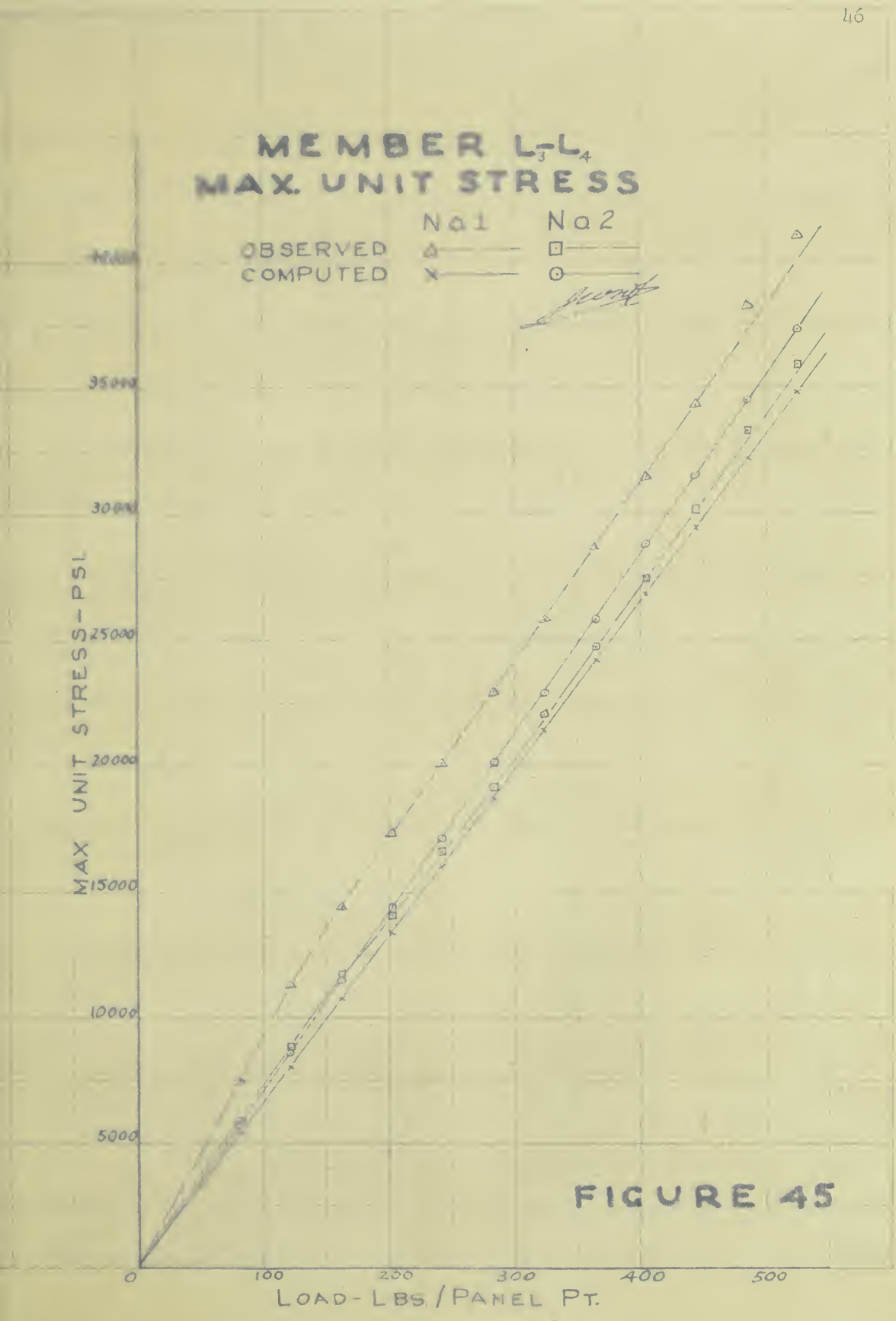

MEMBER L_3-L_4
MAX. UNIT STRESS
No 1
No 2
OBSERVED
COMPUTED
35000
30000
25000
20000
15000
10000
5000
0
MAX UNIT STRESS-PSI
100
200
300
400
500
LOAD-LBS./PANEL PT.

FIGURE 45

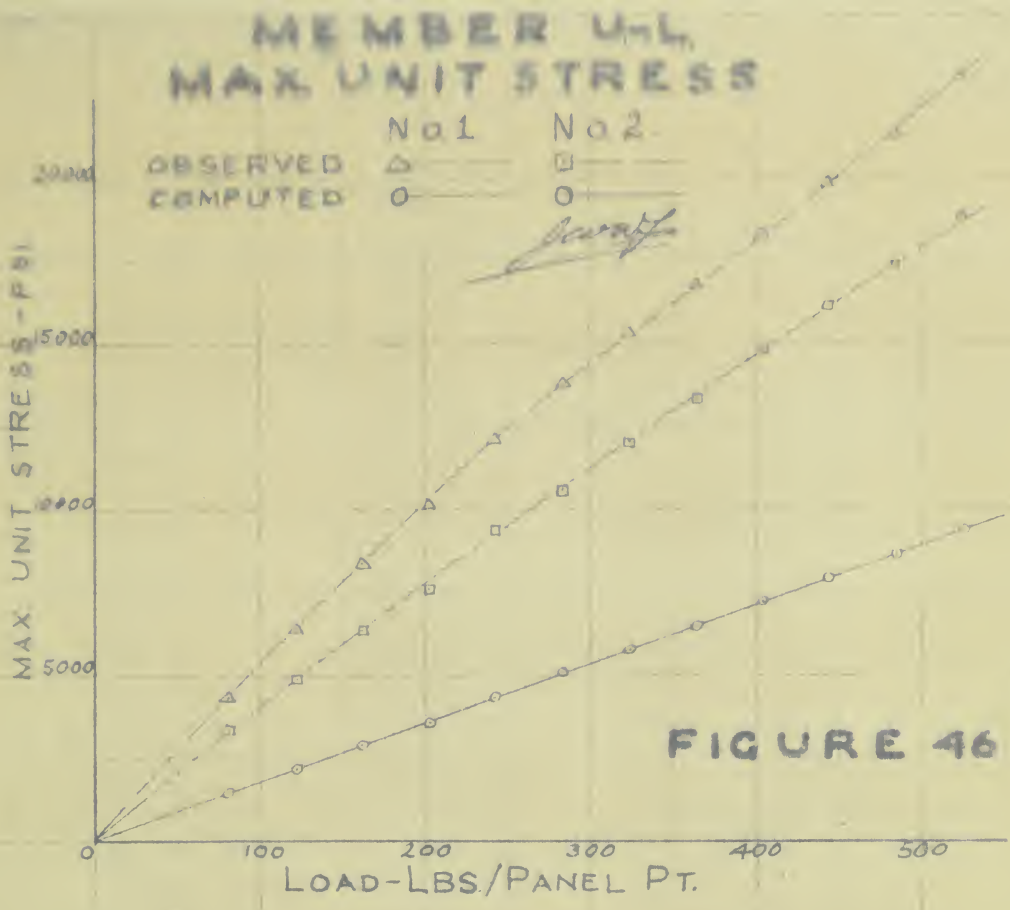


FIGURE 46

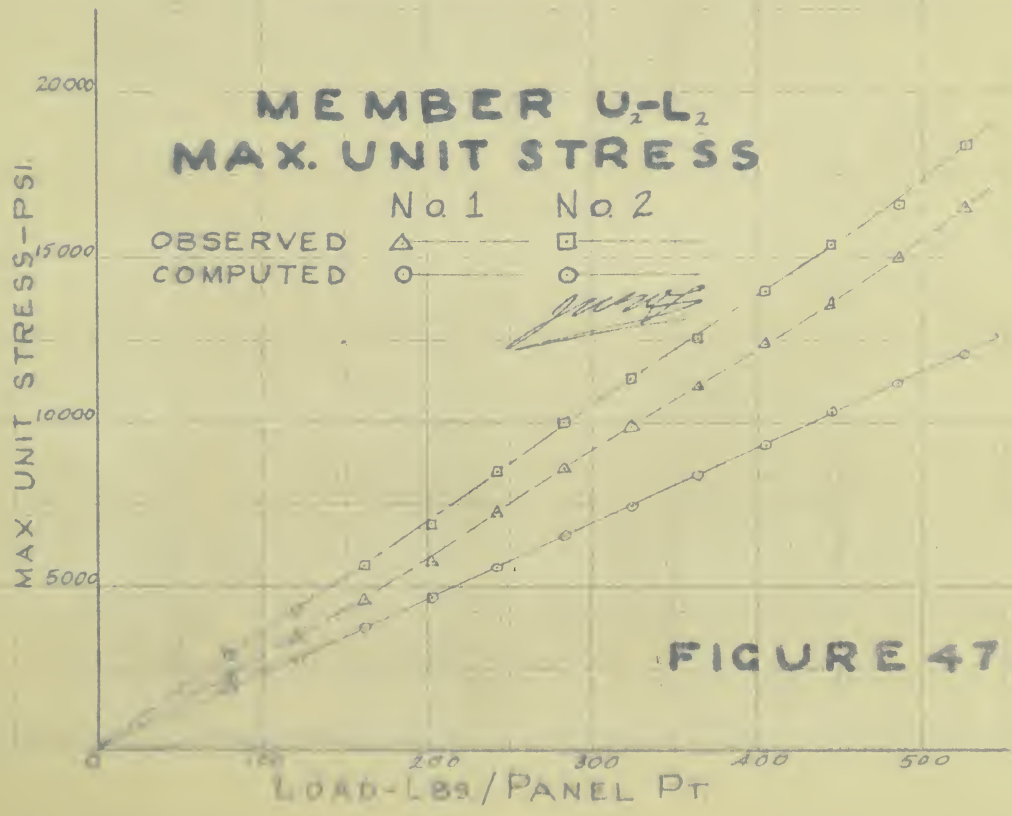


FIGURE 47

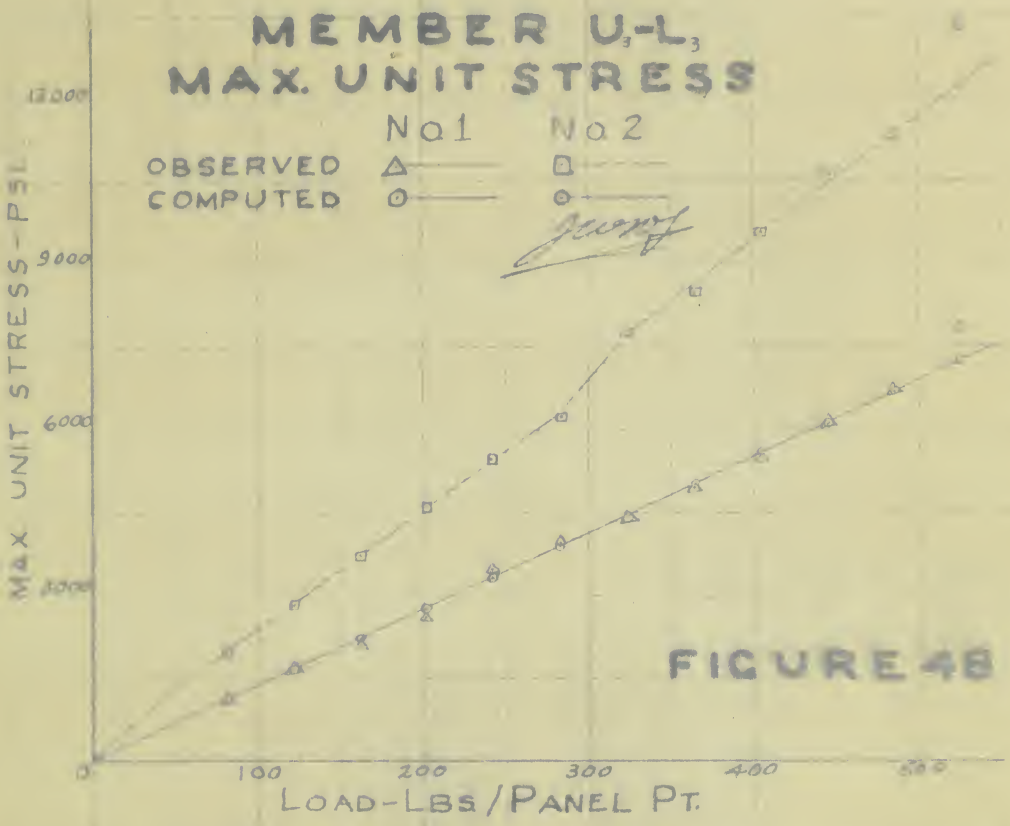


FIGURE 48

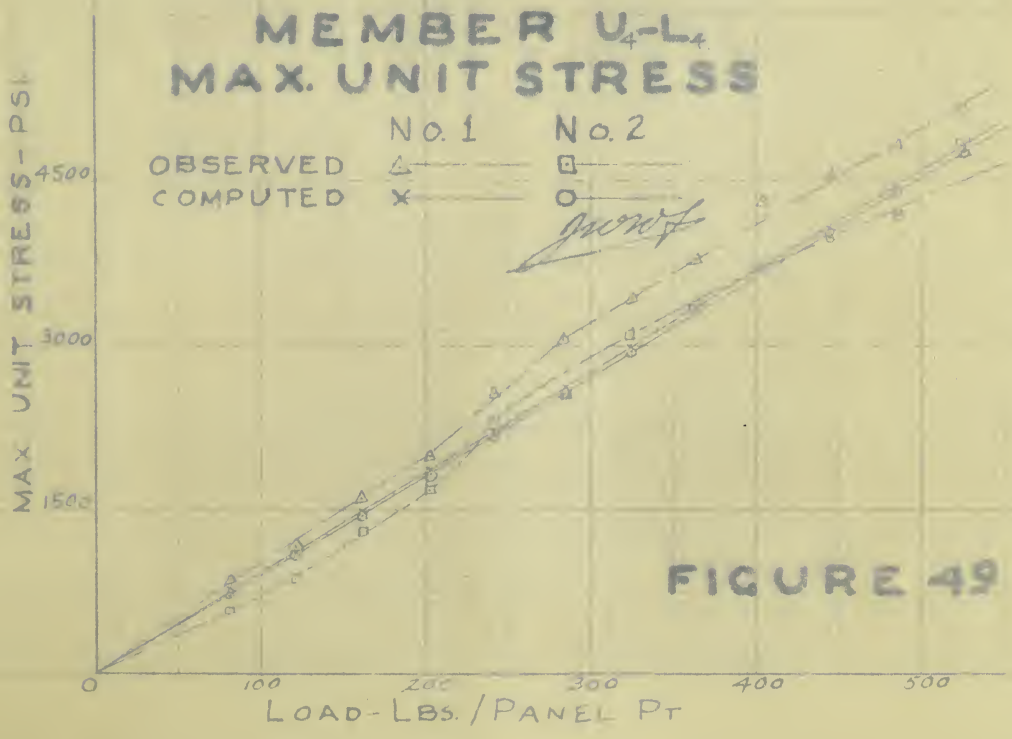


FIGURE 49

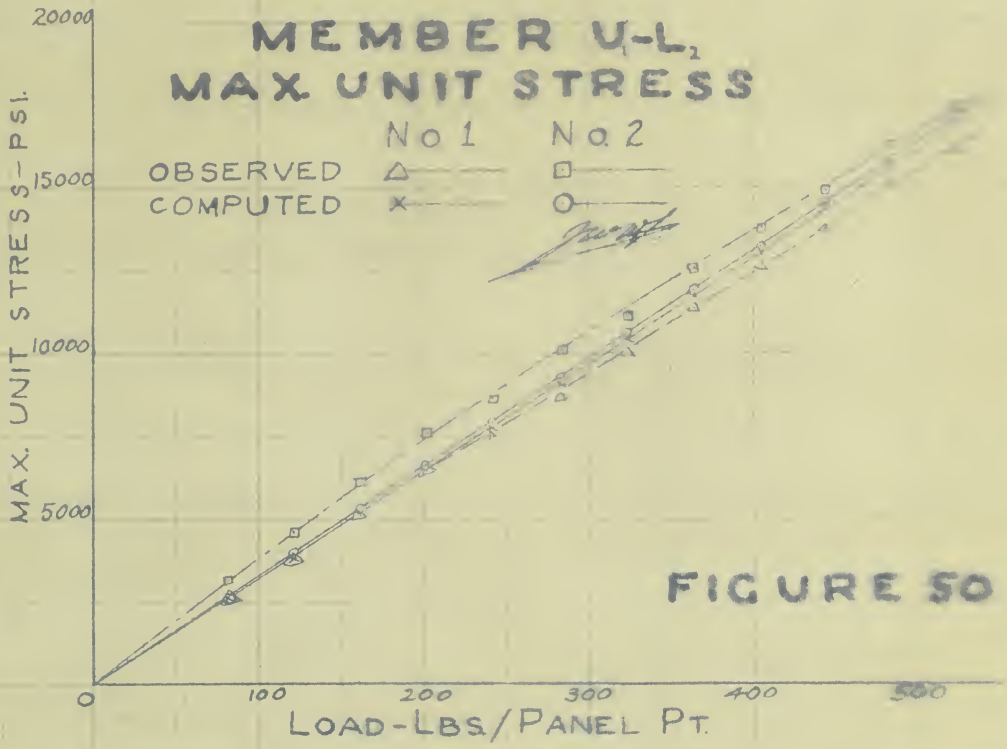


FIGURE 50

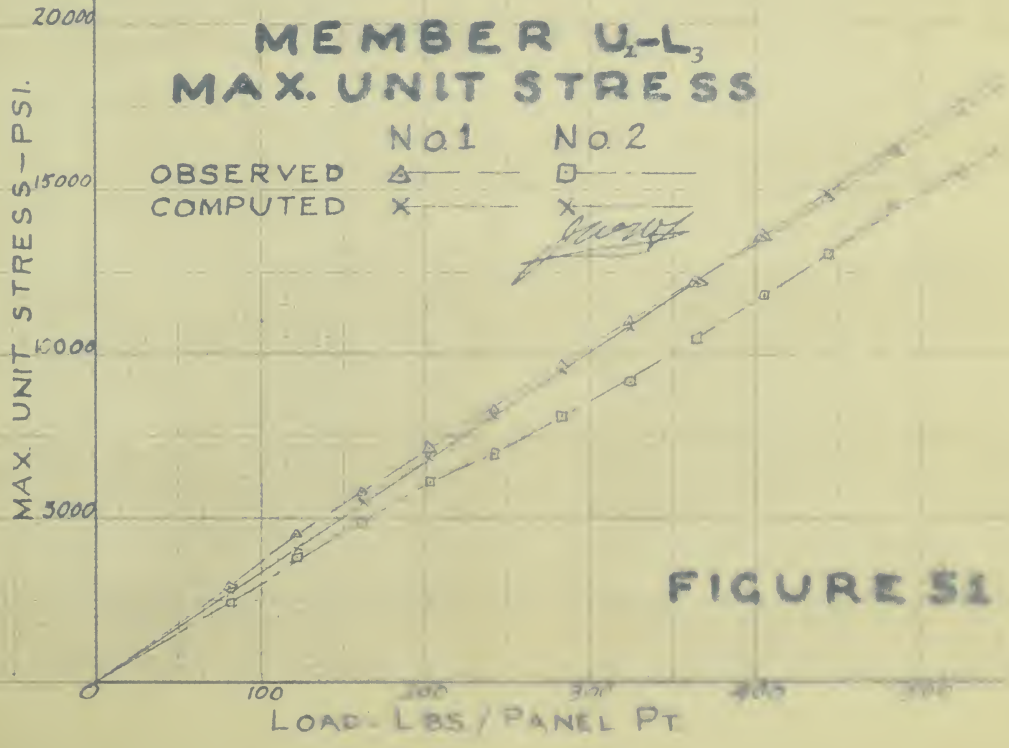


FIGURE 51

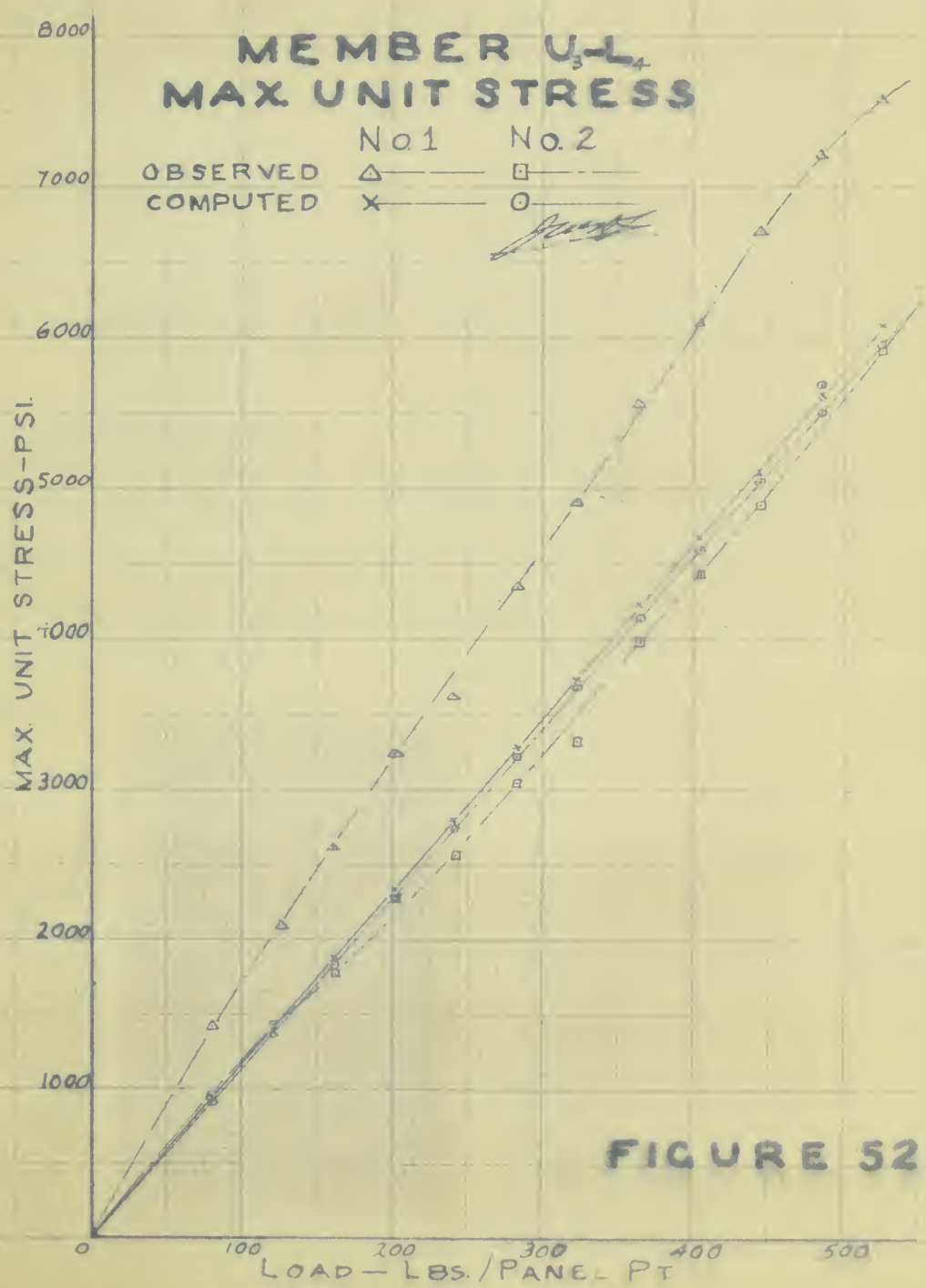

MEMBER U_3–L_4
MAX. UNIT STRESS
No. 1
No. 2
OBSERVED
COMPUTED
8000
7000
6000
5000
4000
3000
2000
1000
0
MAX. UNIT STRESS–PSI.
100
200
300
400
500
LOAD — LBS. / PANEL PT

FIGURE 52

3. Deflections

An examination of the deflection data shows a maximum deflection of, approximately, twenty-eight one hundredths of an inch at the design load. On the basis of one three hundred sixtieth of the span, the allowable deflection would be, approximately, fifty-three one hundredths of an inch or one hundred and eighty-nine per cent of the actual. This is, of course, eminently satisfactory. As a matter fact, the allowable deflection was not exceeded until a load of two hundred and forty-two pounds per lineal foot, or one hundred and sixty-one per cent of the design load, was applied.

4. Method of Failure

The final method of failure of the trusses is, of course, of considerable interest. In Figure 53, which is a pictorial plan view of the collapsed structure, the final position of the component members is well illustrated. This is due to a failure, of the top chord, by buckling.

Fig. 53
Pictorial Plan View After Failure

The mathematical analysis of the buckling load for the top chord is given in Appendix I. It will be noted that the ultimate buckling load, expressed in terms of the total stress in member U_3-U_4, was 20,200 pounds. At the same time, the maximum recorded total stress in member U_3-U_4 was 8,580 pounds for Truss No.1 and 8,170 pounds for Truss No. 2. With the addition of the fourteenth course of brick which caused the collapse, these values could be expected to become, approximately, 9,200 pounds and 8,800 pounds, respectively, both values being considerably less than one-half the computed buckling load.

In order to understand what happened, it is necessary to realize that the support, against buckling, which is afforded the upper chord by the vertical members is an elastic force arising from the cantilever action of the vertical. The vertical is fixed at the lower chord joint by the tension in that chord.

As may be seen in the calculations in Appendix I, the critical value of the total stress in member U_3-U_4, if no support is provided by the verticals, is 2,470 pounds. In both cases, this value was nearly reached after a load of one hundred sixty-two pounds per panel point had been applied to the truss. When this stage was reached, it was necessary for the upper chord to buckle a slight amount before the elastic force of the cantilever columns, or verticals, could take effect. In other words, a horizontal thrust was applied to the upper end of the verticals at this point. As the load continued to increase, the magnitude of this horizontal thrust also continued to increase. However, the vertical members also carried compression loads which were applied to the ends of the member and thus the net effect of the verticals was that of a series of columns, fixed at one end and free at the other, resisting vertical and horizontal

components of loading. The deformation curve of this type of column is well known. The critical load is given by the Euler Equation

$P_{cr} = \frac{\pi^2 EI}{4\ell^2}$, assuming buckling due solely to a vertical, axial load. This gives a critical load of 600 pounds for the three-eighth inch diameter members and 1,940 pounds for the one-half inch diameter members.

Now, noting the values of the total stresses in members U_2-L_2 and U_3-L_3, at the last loading before failure occurred, we find that they were 1,120 pounds and 825 pounds for Truss No. 1 and 1,170 pounds and 745 pounds for Truss No. 2. These values are well over the critical value even if the additional bending stresses are considered to have no effect. Thus, these members would no longer be capable of supplying support to the upper chord. With these members disregarded the total stress in U_3-U_4 at buckling becomes 9,920 pounds.

Let us also consider the observed total stress in the members U_1-L_1 at the same loading. For Truss No.1 this value is 1,690 pounds while for Truss No.2 it is 1,800 pounds. These values, in view of the excessive eccentricities in joint L_1 are very close to being critical, thus leaving only one elastic support, namely, U_4-L_4. The value of the buckling load for U_3-U_4 now becomes 7,440 pounds, or considerably less than the observed values.

Since failure occurred somewhere between the last two cases, the explanation appears to be that the final increment of load caused the members U_1-L_1 to collapse, thus allowing the upper chord to buckle, since it was then overstressed. The fact that the members were welded would probably introduce some slight degree of restraint at the supposedly free ends of the verticals thus raising their

critical loads. At the same time the bending stresses present in these same members would act to lower the critical load. In view of the indeterminate character of these factors, the check between actual and theoretical values clearly justifies the explanation offered.

CONCLUSIONS

1. The theory of simply-connected trusses has been very well substantiated by the satisfactory comparison of theoretical and actual total stresses, since the amount of shear which would be absorbed by the chord members in this case would be very small. Hence, the total stresses should be very nearly the same as in the simply-connected theory.

2. The wide variation of secondary bending stresses gives very inconclusive evidence, but it certainly indicates that proper care in the removal of eccentricities at the lower chord joints would be beneficial, particularly in joint L_1.

3. There is also fairly conclusive evidence that the method of attaching the web members to the upper chord is of material benefit to the latter.

4. The deflections of the truss are very satisfactory.

5. The overall factor of safety of the trusses is in the neighborhood of 1.75 which is quite satisfactory.

6. The verticals are apt to fail as columns fixed at one end and free at the other instead of as pin-ended columns. This would tend to indicate that the working stresses should be one-quarter of those normally used in this work. However, there is a good indication that the member L_3-L_4 had exceeded the elastic limit at approximately the same loading as caused buckling. Therefore, it is doubtful that the strength of the truss would be materially increased by such a practice.

7. With the type of joint being used at L_1, a very liberal factor of safety should be applied in the design of member U_1-L_1.

APPENDIX I

DATA AND SAMPLE CALCULATIONS

Notation

A -- Cross-sectional area

Bending Stress -- Unit bending stress at gauge

Bot. -- Bottom gauge

Comp. Strain -- Unit axial compression strain

Comp. Stress -- Unit axial compression stress

Dia. -- Diameter

Difference -- Cumulative difference between gauge reading and zero reading

E -- Young's Modulus of Elasticity

f -- Fibre stress due to bending

I -- Moment of inertia about neutral axis

I_g -- Moment of inertia about centre of gravity axis

I_i -- Moment of inertia about axis i-i

Load -- Load per panel point

M -- Bending moment

Mean Strain -- Unit axial strain

P -- Axial load or total stress

R -- Resistance

Readings -- observed gauge readings

S -- Section Modulus

Tension Strain -- Unit axial tension strain

Tension Stress -- Unit axial tension stress

Total Stress -- Total axial stress

$\bar{x}$ -- Distance to centre of gravity in x direction

$\bar{y}$ -- Distance to centre of gravity in y direction

y -- Distance from neutral axis to fibre considered

Ultimate -- Ultimate strength

It is important to note that due to the construction of the test members, the accuracy of the observed results can be depended upon only to two figures. However, the results have been carried to three figures on the assumption that it would give a closer approach to the correct answer.

A value of $E = 29 \times 10^6$ psi. was used for all calculations.

SAMPLE CALCULATIONS

Properties of Angle Sections

Since these were equal-legged angles, the calculations were merely a matter of finding the moment of inertia about a horizontal axis through the centre of gravity. This was based upon the assumption that bending took place about this axis.

Determination of Total Stress

(1) Given: Truss No. 1
Member L_3-L_4
Load = 323 lbs.
Tension strain = 733 micro.in.
A = 0.250 sq.in.

Tension Stress = $733 \times 10^6 \times 29 \times 10^6$ = 21,200 psi.

Total Stress = 21,200 x 0.250 = 5,300 lbs.

(2) Given: Truss No.1
Member U_2-U_3
Load = 323 lbs.
A = 0.693 sq.in.
Distance to upper gauge from neutral axis = 0.221 in.
Distance to lower gauge from neutral axis = 0.482 in.
Difference - Upper = 326 micro.in.
Difference - Lower = 81 micro.in.
Differential between upper and lower = 245 micro.in.

Ratio of bending strain at lower to upper = $\frac{482}{221}$ = 2.18

Bending strain at upper gauge = $\frac{245}{3.18}$ = 77 micro.in.

Comp. Strain = 326 - 77=249

Comp. Stress = 7,220 psi.

Total Stress = 7,220 x 0.693 = 5,000 lbs.

Determination of Bending Stresses

(1) Given: Truss No. 1
Member L_3-L_4
Load = 323 lbs.
Differential strain = 318 micro.in.

Bending strain = $\frac{318}{2}$ = 159 micro.in.

Bending stress = $159 \times 10^{-6} \times 29 \times 10^{6}$ = 4,610 psi.

(2) Given: Truss No. 1
Member U_2-U_3
Load = 323 lbs.
Bending strain at upper gauge = 77 micro.in.

Bending stress = 77 x 29 = 2,230 psi.

Note: The maximum stress always occurred at the upper gauge.

Determination of Observed Maximum Unit Stress

(1) Given: Truss No. 1
Member L_3-L_4
Load = 323 lbs.
Axial unit stress = 21,200 psi.
Bending stress = 4,610 psi.

Maximum unit stress = 21,200 + 4,600 = 25,800 psi.

(2) Given: Truss No. 1
Member U_2-U_3
Load = 323 lbs.
Axial unit stress = 7,220 psi.
Bending stress at upper gauge = 2,230 psi.

Bending stress at upper fibre = $\frac{0.398}{0.221} \times 2,230$ = 4,010 psi.

Maximum unit stress = 7,220 + 4,010 = 11,200 psi.

Determination of Actual Computed Total Stresses

This was determined simply by the use of statics assuming axial loads and simple connections. The actual dimensions of the trusses were used for determining slopes of members.

Determination of Computed Maximum Unit Stresses

(1) Given: Truss No. 1
Member L_3-L_4
Load = 323 lbs.
P = 5,360 lbs.
A = 0.250 sq. in.

Computed maximum unit stress = $\frac{5,360}{0.250}$ = 21,400 psi.

(2) Given: Truss No. 1
Member U_2-U_3
Load = 323 lbs.
Length = 24.1 in.
Distance from gauge to panel point = 12.2 in. or 11.9 in.
w = load per lineal inch
S = 0.268 $in.^3$
P = 5,360 lbs.

$$M = \frac{wlx}{2} - \frac{wx^2}{2}$$

$$= \frac{323}{24.1} \cdot \frac{24.1}{2} \cdot 11.9 - \frac{323}{24.1} \cdot \frac{11.9}{2} \cdot 11.9$$

$$= 1920 - 950 = 970 \text{ in. lbs.}$$

$$f = \frac{970}{0.268} = 3,620 \text{ psi. at top gauge}$$

$$= 3,620 \times \frac{398}{221} = 6,540 \text{ psi. at top fibre}$$

Axial stress = $\frac{5,360}{0.693}$ = 7,740 psi.

Computed maximum unit stress = 7,740 + 6,540 = 14,300 psi.

TABLE No.1

PROPERTIES OF GAUGE SECTIONS

Member	Ave. Thick. in.	Ave. Width in.	A. Sq.in.	y in.	I $in.^4$	S $in.^3$
Truss No.1						
U_0-L_1	0.2513	0.9729	0.245	0.126	0.00129	0.0102
L_1-L_2	0.2498	0.9951	0.249	0.125	0.00129	0.01035
L_2-L_3	0.2507	0.9964	0.250	0.125	0.00131	0.0104
L_3-L_4	0.2501	0.9980	0.250	0.125	0.00130	0.0104
Truss No.2						
U_0-L_1	0.2499	0.9711	0.243	0.125	0.00126	0.0101
L_1-L_2	0.2454	0.9957	0.244	0.123	0.00123	0.00999
L_2-L_3	0.2394	0.9943	0.238	0.120	0.00114	0.00950
L_3-L_4	0.2399	0.9937	0.238	0.120	0.00114	0.00953

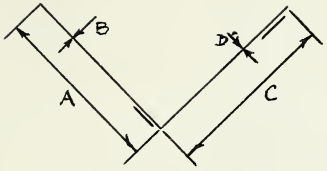
Member	Ave. Diam. in.	A. Sq.in.	y in.	I $in.^4$	S $in.^3$
Truss No.1					
U_1-L_1	0.4990	0.196	0.250	0.00304	0.0122
U_1-L_2	0.4941	0.192	0.247	0.00293	0.0119
U_2-L_2	0.3713	0.108	0.187	0.000930	0.00500
U_2-L_3	0.3705	0.108	0.185	0.000930	0.00503
U_3-L_3	0.3726	0.109	0.186	0.000946	0.00508
U_3-L_4	0.3693	0.107	0.185	0.000913	0.00494
U_4-L_4	0.3725	0.109	0.186	0.000945	0.00508

TABLE No. 1 (Cont'd)

PROPERTIES OF GAUGE SECTIONS

Member	Ave. Diam. in.	A. Sq.in.	y in.	I $in.^4$	S $in.^3$
Truss No.2					
U_1-L_1	0.4993	0.196	0.250	0.00305	0.0122
U_1-L_2	0.5010	0.197	0.251	0.00308	0.0123
U_2-L_2	0.3712	0.108	0.186	0.000930	0.00500
U_2-L_3	0.3752	0.110	0.188	0.000966	0.00519
U_3-L_3	0.3728	0.109	0.187	0.000948	0.00507
U_3-L_4	0.3739	0.110	0.187	0.000962	0.00514
U_4-L_4	0.3748	0.110	0.188	0.000966	0.00519

Note: All gauges on upper chord set $\frac{1}{4}$" from the nearest edge.



Member	Ave. A in.	Ave. B in.	Ave. C in.	Ave. D in.	A. $in.^2$	I $in.^4$	S_{Top} $in.^3$	$S_{Bot.}$ $in.^3$
Truss No.1								
U_0-U_1	1.5038	0.2527	1.4910	0.2502	0.690	0.0586	0.269	0.1215
U_1-U_2	1.5063	0.2528	1.4878	0.2501	0.690	0.0584	0.269	0.1215
U_2-U_3	1.5101	0.2532	1.4948	0.2504	0.693	0.0592	0.268	0.123
U_3-U_4	1.5023	0.2534	1.4933	0.2508	0.691	0.0589	0.266	0.122

TABLE No. 1 (Cont'd)

PROPERTIES OF GAUGE SECTIONS

Member	Ave. A in.	Ave. B in.	Ave. C in.	Ave. D in.	A. $in.^2$	I $in.^4$	S_{Top} $in.^3$	$S_{Bot.}$ $in.^3$
Truss No.2								
U_0-U_1	1.5030	0.2540	1.4885	0.2490	0.689	0.0586	0.268	0.122
U_1-U_2	1.5117	0.2537	1.4997	0.2500	0.695	0.0598	0.272	0.123
U_2-U_3	1.5106	0.2549	1.4959	0.2503	0.695	0.0596	0.271	0.123
U_3-U_4	1.5133	0.2539	1.4919	0.2475	0.691	0.0592	0.273	0.122

TABLE No. 2

GAUGE APPLICATION DATA

NOTE: R's include the resistance of the leads (0.078 ohms)

Gauge No.	'R' Before Application Ohms	'R' After Application Ohms	Leakage 'R' Megohms	Location
	Type A3 Factor = 2.05 Lot MZ			
1	119.33	120.00	50	#1 W U_4-L_4
2	118.94	119.70	50	#1 E U_4-L_4
3	119.41	119.91	50	#2 E U_4-L_4
4	119.05	119.61	50	#2 W U_4-L_4
5	119.33	119.91	50	#2 E U_3-L_3
* 6	118.36	--	--	--
7	119.23	119.82	50	#2 E U_2-L_2
8	119.27	119.91	50	#2 W U_2-L_2
9	119.14	119.77	50	#2 Bot. U_4-L_4
10	119.07	119.64	50	#2 Top U_3-L_4
11	119.02	119.64	50	#2 W U_3-L_3
12	119.30	119.91	50	#2 Bot. U_2-L_3
13	119.64	120.36	50	#2 Top U_2-L_3
14	119.11	119.82	50	#1 E U_2-L_2
15	119.25	119.91	50	#1 W U_2-L_2
16	119.41	120.00	50	#1 Bot. U_2-L_3
17	119.00	119.86	50	#1 Top U_2-L_3
18	119.36	119.91	50	#1 E U_3-L_3
19	119.55	120.09	50	#1 W U_3-L_3
* 20	119.59	120.05	0	#1 Bot. U_3-L_4

TABLE No. 2 (Cont'd)

GAUGE APPLICATION DATA

Gauge No.	'R' Before Application Ohms	'R' After Application Ohms	Leakage 'R' Megohms	Location
21	119.45	120.05	50	#1 Top U_3-L_4
22	119.55	120.00	50	#2 E U_1-L_1
23	119.91	120.41	50	#2 W U_1-L_1
24	119.68	120.32	50	#2 Bot. U_1-L_2
25	119.64	120.27	50	#2 Top U_1-L_2
26	119.68	120.27	50	#1 E U_1-L_1
27	119.84	120.43	50	#1 Bot. U_1-L_2
28	119.50	120.14	0.1	#1 W U_1-L_1
29	119.86	120.43	50	#1 Top U_1-L_2
73	119.52	120.00	50	Replace No.20
74	119.61	120.09	50	Replace No.28
75	119.05	119.59	50	Compensating
77	119.14	119.68	50	Replace No.15
	Type A1 Factor = 2.05 Lot MY			
30	119.61	120.16	50	#2 Bot. U_0-L_1
31	119.98	120.16	50	#2 Top U_0-L_1
32	119.66	120.00	50	#2 Bot. L_1-L_2
33	119.55	120.00	50	#2 Top L_1-L_2
34	119.84	120.09	50	#2 Bot. L_2-L_3
35	120.00	120.25	50	#2 Top L_2-L_3
36	119.70	120.05	50	#2 Bot. L_3-L_4
* 37	119.77	120.25	0	#2 Top L_3-L_4

TABLE No. 2 (Cont'd)

GAUGE APPLICATION DATA

Gauge No.	'R' Before Application Ohms	'R' After Application Ohms	Leakage 'R' Megohms	Location
38	119.68	120.00	50	#1 Bot. U_0-L_1
39	119.98	120.27	50	#1 Top U_0-L_1
*40	119.77	120.20	30	#1 Bot. L_1-L_2
41	119.66	120.05	50	#1 Top L_1-L_2
42	119.80	120.41	50	#1 Bot. L_2-L_3
43	120.00	120.11	50	#1 Top L_2-L_3
44	119.75	120.20	50	#1 Bot. L_3-L_4
45	119.82	120.20	50	#1 Top L_3-L_4
46	119.84	120.14	50	#2 Lower U_0-U_1
*47	Damaged			#2 Upper U_0-U_1
48	119.73	120.00	50	#2 Lower U_1-U_2
49	119.91	120.14	50	#2 Upper U_1-U_2
50	119.57	120.00	50	#2 Lower U_2-U_3
51	119.68	120.16	50	#2 Upper U_2-U_3
52	120.07	120.36	50	#2 Lower S. U_3-U_4
53	119.66	120.05	50	#2 Upper S. U_3-U_4
54	119.64	120.09	50	#2 Lower N. U_3-U_4
55	119.86	120.18	50	#2 Upper N. U_3-U_4
56	119.84	120.07	50	#1 Lower U_0-U_1
57	119.91	120.52	50	#1 Upper U_0-U_1
58	119.98	120.41	50	#1 Lower U_1-U_2
59	119.36	119.66	50	#1 Upper U_1-U_2

TABLE No. 2 (Cont'd)

GAUGE APPLICATION DATA

Gauge No.	'R' Before Application Ohms	'R' After Application Ohms	Leakage 'R' Megohms	Location
60	119.61	119.68	50	#1 Lower U_2-U_3
61	119.82	120.18	50	#1 Upper U_2-U_3
62	119.77	120.09	50	#1 Lower S. U_3-U_4
63	119.77	119.98	50	#1 Upper S. U_3-U_4
64	119.80	120.05	50	#1 Lower N. U_3-U_4
65	120.02	120.45	50	#1 Upper N. U_3-U_4
66	119.93	120.32	50	Replace No.47
67	119.75	120.23	50	Replace No.37
68	120.09	120.18	50	Replace No.40
69	119.98	120.25	50	Cross Brace
70	119.48	120.09	50	Cross Brace
71	120.05	120.11	50	Cross Brace
72	119.68	119.80	50	Cross Brace
76	119.84	120.09	50	Compensating

* NOTE: Gauge replaced.

TABLE No. 3

WEIGHTS OF BRICKS

No. of Bricks	Weight Lbs.	Ozs.	No. of Bricks	Weight Lbs.	Ozs.
1	4	12	1	4	14
2	9	10	2	9	11.5
3	14	6.5	3	14	12
4	19	6	4	19	7
5	24	9	5	24	5
6	29	3	6	29	1.5
7	34	9.5	7	33	12.5
8	39	11.5	8	38	8
9	44	11	9	43	8.5
10	49	8	10	48	9
11	54	4	11	53	5.5
12	59	4.5	12	58	7
13	64	0.75	13	63	4
14	68	13.5	14	68	0
15	73	14	15	73	0
17	83	9.75			
18	88	8			
19	93	7.75			
20	98	6.75			
21	103	4.5			
22	108	1			
23	112	15			
24	117	12			
25	122	13			

Weight of 40 Bricks = 196 lbs.
1 Course = 132 Bricks = 647 lbs.
= 647 ÷ 32 = 20.2 lbs./ft/ of truss = 40.4 lbs./panel point

At end of test, 40 bricks weighed 196 lbs.

TABLE No.4

SWITCHES
Lower C1-17--Upper C1-18

Truss No.1
Member U_0-U_1

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings C1-17	Readings C1-18	Difference C1-17	Difference C1-18	Comp. Strain	Comp. Stress	Bending Stress C1-18	Total Stress
	Stage 4	Stage 6						
0	1659	1413	0	0	0	0	0	0
81	1713	1313	+54	-100	52	1510	1390	1040
121	1739	1279	+80	-134	67	1940	1940	1340
162	1764	1250	+105	-163	79	2290	2420	1580
202	1789	1218	+130	-195	94	2730	2930	1880
242	1792	1180	+133	-233	119	3450	3300	2380
283	1800	1151	+141	-262	136	3940	3650	2720
323	1823	1139	+164	-274	137	3980	3970	2740
364	1826	1111	+167	-302	156	4520	4240	3120
404	1832	1088	+173	-325	170	4930	4490	3400
444	1857	1072	+198	-341	173	5020	4870	3460
485	1870	1059	+211	-354	178	5160	5100	3560
525	1874	1030	+215	-383	197	5710	5390	3940
566	Failure							

TABLE No.5

SWITCHES
Lower I-1--Upper I-2

Truss No.2
Member U_0-U_1

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings I-1	Readings I-2	Difference I-1	Difference I-2	Comp. Strain	Comp. Stress	Bending Stress I-2	Total Stress
	Stage 4	Stage 5						
0	1880	1715	0	0	0	0	0	0
81	1889	1674	+9	-41	25	725	464	500
121	1894	1657	+14	-58	35	1020	667	700
162	1901	1641	+21	-74	44	1280	870	870
202	1904	1628	+24	-87	52	1510	1020	1040
242	1894	1593	+14	-122	79	2290	1250	1580
283	1890	1580	+10	-135	90	2610	1305	1800
323	1899	1568	+19	-147	95	2760	1510	1900
364	1898	1547	+18	-168	110	3190	1680	2200
404	1901	1530	+21	-185	120	3480	1890	2400
444	1911	1518	+31	-197	126	3650	2090	2520
485	1915	1499	+35	-216	137	3980	2290	2740
525	1912	1478	+32	-237	153	4440	2440	3060
566	Failure							

TABLE No.6

SWITCHES
Lower B2-10--Upper B2-11

Truss No.1
Member U_1-U_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings B2-10	Readings B2-11	Difference B2-10	Difference B2-11	Comp. Strain	Comp. Stress	Bending Stress B2-11	Total Stress
	Stage 5	Stage 2						
0	1961	1889	0	0	0	0	0	0
81	1923	1829	38	60	53	1540	203	1060
121	1908	1799	53	90	78	2260	334	1560
162	1892	1768	69	121	105	3050	464	2100
202	1875	1731	86	158	136	3940	638	2720
242	1850	1685	111	204	175	5070	840	3500
283	1837	1648	124	241	205	5950	1040	4110
323	1832	1622	129	267	224	6500	1250	4580
364	1817	1587	144	302	253	7340	1420	5070
404	1804	1560	157	329	275	7980	1570	5510
444	1799	1537	162	352	293	8500	1710	5860
485	1790	1510	171	379	314	9110	1890	6290
525	1772	1474	189	415	345	10000	2030	6900
566	Failure							

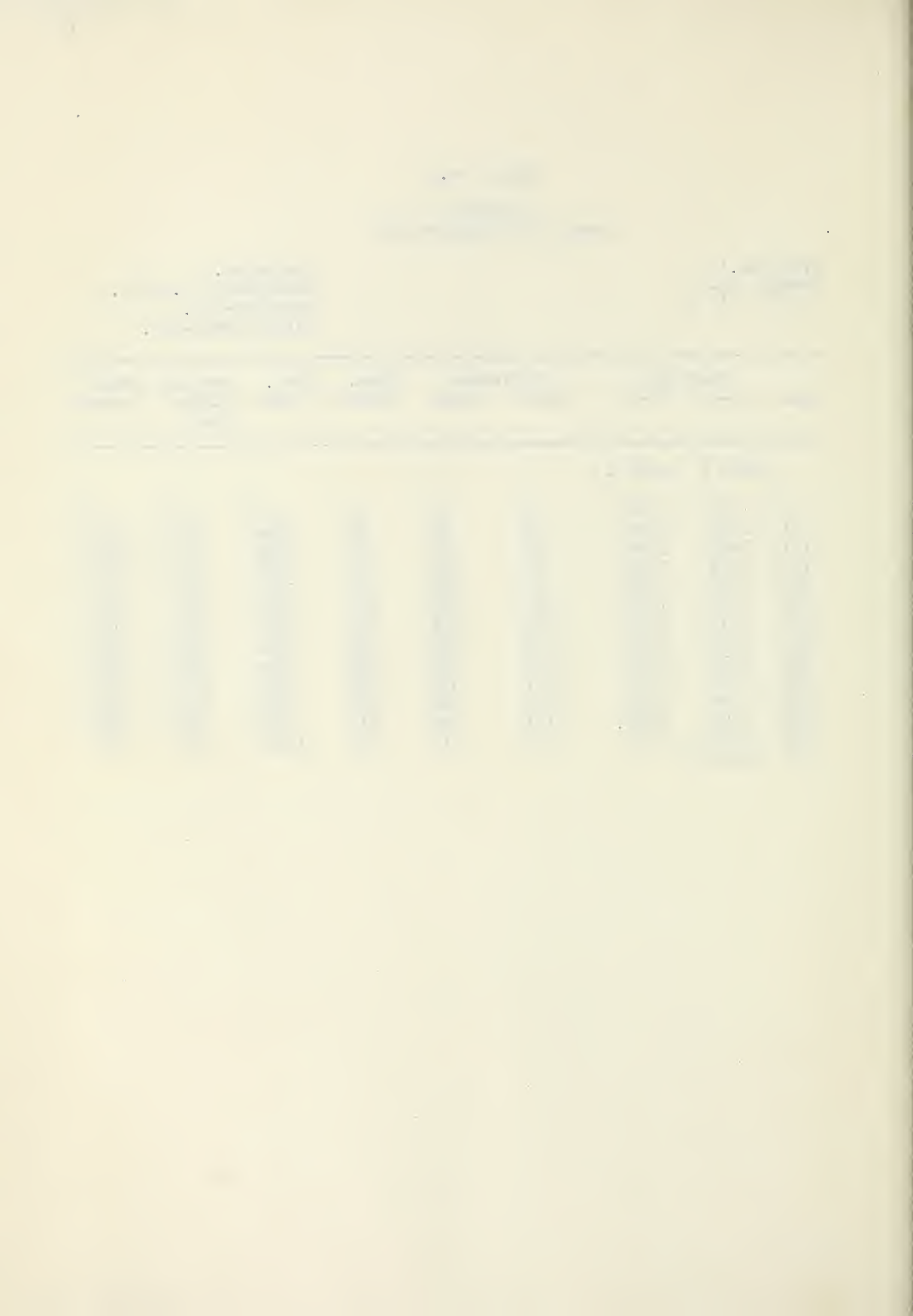


TABLE No.7

SWITCHES
Lower I-3--Upper I-4

Truss No.2
Member U_1-U_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings I-3	Readings I-4	Difference I-3	Difference I-4	Comp. Strain	Comp. Stress	Bending Stress I-4	Total Stress
	Stage 4	Stage 4						
0	1347	1912	0	0	0	0	0	0
81	1329	1850	18	62	48	1390	406	970
121	1303	1823	44	89	75	2180	406	1510
162	1293	1799	54	113	95	2760	522	1920
202	1287	1772	60	140	115	3340	725	2320
242	1261	1724	86	188	156	4520	928	3140
283	1252	1692	95	220	181	5250	1130	3650
323	1244	1672	103	240	197	5710	1250	3960
364	1227	1637	120	275	228	6610	1360	4590
404	1220	1608	127	304	249	7110	1590	4940
444	1214	1584	133	328	267	7740	1770	5380
485	1205	1558	142	354	288	8350	1910	5800
525	1191	1526	156	386	314	9110	2090	6330
566	Failure							

TABLE No.8

SWITCHES
Lower B2-12--Upper 2-1-9

Truss No.1
Member U_2-U_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings B2-12	Readings 2-1-9	Difference B2-12	Difference 2-1-9	Comp. Strain	Comp. Stress	Bending Stress 2-1-9	Total Stress
	Stage 4	Stage 5						
0	1132	1074	0	0	0	0	0	0
81	1111	992	21	82	63	1840	551	1270
121	1104	951	28	123	93	2700	870	1870
162	1099	915	33	159	119	3450	1160	2390
202	1089	871	43	203	153	4440	1450	3080
242	1067	821	65	253	194	5630	1710	3900
283	1052	780	80	294	227	6580	1940	4560
323	1051	748	81	326	249	7220	2230	5000
364	1052	702	100	372	286	8290	2500	5740
404	1029	664	103	410	313	9080	2810	6290
444	1028	648	104	426	325	9430	2930	6530
485	1023	600	109	474	350	10400	3340	7210
525	1013	564	119	510	387	11200	3570	7770
566	Failure							

TABLE No.9

SWITCHES
Lower I-5--Upper I-6

Truss No.2
Member U_2-U_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings I-5	Readings I-6	Difference I-5	Difference I-6	Comp. Strain	Comp. Stress	Bending Stress I-6	Total Stress
	Stage 4	Stage 4						
0	1328	1949	0	0	0	0	0	0
81	1303	1867	25	82	64	1860	942	1290
121	1298	1833	30	116	89	2580	1410	1790
162	1281	1810	47	139	110	3190	1520	2220
202	1272	1770	56	179	141	4090	1980	2840
242	1243	1722	85	227	183	5310	2310	3690
283	1237	1680	91	269	213	6170	2900	4290
323	1227	1651	101	298	237	6870	3200	4770
364	1210	1609	118	340	274	7950	3450	5520
404	1202	1572	126	377	299	8670	4080	6020
444	1198	1541	130	408	321	9310	4550	6470
485	1188	1503	140	446	351	10180	4980	7070
525	1180	1477	148	472	371	10750	5290	7470
566	Failure							

TABLE No. 10

SWITCHES

Lower South 2-1-10 Upper South 2-2-1 Lower North 2-2-2 Upper North 2-2-3

Truss No. 1
Member U_3-U_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings				Difference				Comp Stress	Bending Stress	Total Stress
	2-1-10	2-2-1	2-2-2	2-2-3	2-1-10	2-2-1	2-2-2	2-2-3		2-2-3	
	Stage 4	Stage 4	Stage 4	Stage 6							
0	1741	1170	1530	1377	0	0	0	0	0	0	0
81	1691	1095	1484	1302	50	75	46	75	1940	261	1340
121	1669	1058	1465	1269	72	112	65	108	2815	377	1940
162	1648	1020	1443	1230	93	150	87	147	3770	551	2600
202	1618	979	1420	1192	123	191	110	185	4810	666	3320
242	1583	930	1390	1145	158	240	140	232	6070	841	4190
283	1563	889	1370	1101	178	281	160	276	7080	1040	4890
323	1551	851	1353	1070	190	319	177	307	7890	1190	5450
364	1522	809	1330	1027	219	361	200	350	8990	1360	6200
404	1502	773	1310	988	239	397	220	389	9920	1540	6850
444	1494	740	1300	959	247	430	230	418	10600	1710	7340
485	1473	707	1280	922	268	463	250	455	11500	1860	7930
525	1457	670	1262	881	284	500	268	496	12400	2060	8580
566	Failure										

TABLE No. 11

SWITCHES

Lower South I-7 Upper South I-8 Lower North I-9 Upper North I-10

Truss No.2
Member U_3-U_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings				Difference				Comp Stress	Bending Stress I-7	Total Stress
	I-7	I-8	I-9	I-10	I-7	I-8	I-9	I-10			
	Stage 5	Stage 4	Stage 4	Stage 5							
0	1869	1535	1641	1032	0	0	0	0	0	0	0
81	1824	1459	1604	971	45	76	37	61	1740	290	1200
121	1809	1420	1583	940	60	115	58	92	2580	522	1780
162	1787	1380	1571	910	82	155	70	122	3450	696	2380
202	1769	1340	1548	874	100	195	93	158	4380	899	3020
242	1739	1290	1528	831	130	245	113	201	5540	1070	3820
283	1717	1250	1500	789	152	285	141	243	6580	1250	4540
323	1708	1290	1498	750	161	316	143	282	7340	1450	5070
364	1681	1173	1470	708	188	362	171	324	8470	1620	5850
404	1663	1140	1452	681	206	395	189	351	9230	1770	6370
444	1648	1101	1435	649	221	434	206	383	10100	2000	6950
485	1628	1054	1413	612	241	481	228	420	11100	2260	7660
525	1609	1028	1392	580	260	507	249	452	11840	2320	8170
566	Failure										

TABLE No. 12

SWITCHES
Bottom 2-2-4--Top 2-2-5

Truss No.1
Member U_0-L_1

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings 2-2-4	Readings 2-2-5	Difference 2-2-4	Difference 2-2-5	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 5	Stage 6						
0	345	440	0	0	0	0	0	0
81	359	630	14	190	102	2960	2550	725
121	385	710	40	270	155	4490	3340	1100
162	419	780	74	340	207	6000	3860	1470
202	451	849	106	409	258	7480	4400	1830
242	482	899	137	459	298	8640	4670	2120
283	518	960	173	520	347	10700	5040	2620
323	564	1021	219	581	400	11600	5250	2850
364	600	1070	255	630	443	12800	5440	3140
404	642	1122	297	682	490	14200	5590	3480
444	691	1182	346	742	544	15800	5740	3870
485	740	1239	395	799	597	17300	5860	4240
525	787	1292	442	852	647	18800	5940	4600
566	Failure							

TABLE No. 13

SWITCHES
Bottom I-11 Top I-12

Truss No.2
Member U_0-L_1

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings I-11	Readings I-12	Difference I-11	Difference I-12	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 6	Stage 6						
0	33	61	0	0	0	0	0	0
81	149	128	116	67	92	2670	710	649
121	205	164	172	103	138	4000	1000	973
162	262	209	229	148	189	5480	1180	1330
202	318	249	285	188	237	6870	1410	1670
242	369	281	336	220	278	8070	1680	1960
283	415	317	382	256	319	9240	1830	2250
323	480	362	447	301	374	10800	2120	2640
364	526	400	493	339	416	12100	2230	2940
404	583	446	550	385	468	13600	2390	3300
444	648	498	615	437	526	15250	2580	3710
485	692	535	659	474	567	16400	2680	4000
525	738	580	705	519	612	17700	2750	4310
566	Failure							

TABLE No. 14

SWITCHES
Bottom 2-2-6 Top 2-2-7

Truss No.1
Member L_1-L_2

Loads-lbs.
Strains-micro. in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings 2-2-6	Readings 2-2-7	Difference 2-2-6	Difference 2-2-7	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 6	Stage 5						
0	102	477	0	0	0	0	0	0
81	252	517	150	40	95	2760	1600	688
121	320	548	218	71	145	4210	2130	1050
162	380	580	278	103	191	5540	2540	1380
202	433	611	331	134	233	6760	2860	1680
242	479	642	377	165	271	7860	3080	1960
283	529	697	427	220	324	9400	3010	2340
323	582	732	480	255	368	10700	3270	2660
364	624	763	522	286	404	11700	3420	2920
404	671	803	569	326	448	13000	3530	3240
444	724	835	622	358	490	14200	3830	3540
485	775	880	673	403	538	15600	3920	3890
525	821	924	719	447	583	16900	3950	4210
566	Failure							

TABLE No. 15

SWITCHES
Bottom 2-1-4 Top 2-1-5

Truss No.2
Member L_1-L_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings 2-1-4	Readings 2-1-5	Difference 2-1-4	Difference 2-1-5	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 5	Stage 5						
0	389	312	0	0	0	0	0	0
81	555	308	166	-4	81	2350	2470	573
121	632	322	243	10	127	3680	3380	898
162	703	345	314	33	174	5050	4080	1230
202	756	359	367	47	207	6010	4640	1470
242	809	378	420	66	243	7050	5150	1720
283	867	407	478	95	287	8620	5550	2100
323	928	442	539	130	335	9720	5940	2370
364	980	471	591	159	375	10900	6260	2660
404	1037	509	648	197	423	12300	6550	3000
444	1088	544	699	232	466	13500	6780	3300
485	1149	588	760	276	518	15000	7030	3670
525	1198	624	809	312	561	16300	7210	3980
566	Failure							

TABLE No. 16

SWITCHES
Bottom 2-2-8 Top 2-2-9

Truss No.1
Member L_2-L_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings 2-2-8	Readings 2-2-9	Difference 2-2-8	Difference 2-2-9	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 7	Stage 5						
0	69	751	0	0	0	0	0	0
81	241	874	172	123	148	4290	710	1070
121	328	945	259	194	227	6580	940	1640
162	410	1011	341	260	301	8730	1180	2180
202	488	1078	419	327	373	10800	1330	2700
242	558	1140	489	389	439	12750	1450	3180
283	628	1209	559	458	509	14800	1470	3700
323	708	1285	639	534	587	17000	1530	4250
364	775	1349	706	598	652	18900	1570	4720
404	849	1422	780	671	726	21100	1580	5270
444	927	1493	858	742	800	23200	1680	5800
485	1000	1560	931	809	870	25300	1770	6330
525	1072	1648	1003	897	950	27600	1540	6900
566	Failure							

TABLE No. 17

SWITCHES
Bottom 2-1-6 Top 2-1-7

Truss No.2
Member L_2-L_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total Stress-lbs.

Load	Readings 2-1-6	Readings 2-1-7	Difference 2-1-6	Difference 2-1-7	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 5	Stage 6						
0	641	491	0	0	0	0	0	0
81	809	618	168	121	145	4210	680	1000
121	894	689	253	198	226	6560	745	1560
162	977	758	336	267	302	8760	1000	2080
202	1047	817	406	326	366	10600	1160	2560
242	1114	877	473	386	430	12500	1260	2980
283	1192	948	551	457	504	14600	1360	3480
323	1275	1020	634	529	582	16900	1520	4020
364	1351	1089	710	589	654	19000	1630	4510
404	1432	1162	791	671	731	21200	1740	5050
444	1510	1234	869	743	806	23400	1830	5560
485	1599	1318	958	827	893	25900	1900	6150
525	1677	1390	1036	899	968	27100	1990	6440
566	Failure							

TABLE No. 18

SWITCHES
Bottom 2-2-10 Top A1-1

Truss No.1
Member L_3-L_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings 2-2-10	Readings A1-1	Difference 2-2-10	Difference A1-1	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 6	Stage 6						
0	181	160	0	0	0	0	0	0
81	440	270	259	110	185	5370	2160	1340
121	570	341	389	181	285	8270	3020	2070
162	678	415	497	255	376	10900	3510	2720
202	779	491	598	331	465	13500	3870	3380
242	877	564	696	404	550	15900	4230	3980
283	973	645	792	485	639	18500	4450	4620
323	1073	734	892	574	733	21200	4610	5300
364	1170	812	989	652	821	23800	4890	5950
404	1263	898	1082	738	910	26400	4990	6600
444	1362	982	1181	822	1002	29100	5200	7270
485	1500	1077	1319	917	1118	32400	5830	8100
525	1592	1158	1411	998	1205	35000	5990	8750
566	Failure							

TABLE No. 19

SWITCHES
Bottom 2-1-8 Top 2-1-1

Truss No.2
Member L_3-L_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings 2-1-8	Readings 2-1-1	Difference 2-1-8	Difference 2-1-1	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 5	Stage 6						
0	487	307	0	0	0	0	0	0
81	689	466	202	159	181	5250	625	1250
121	790	558	303	251	277	8040	755	1920
162	888	649	401	342	372	10800	855	2570
202	974	730	487	423	455	13200	930	3140
242	1060	808	573	501	537	15600	1050	3720
283	1148	891	661	584	623	18100	1120	4310
323	1245	981	758	674	716	20800	1220	4950
364	1339	1069	852	762	807	23400	1310	5570
404	1433	1161	946	854	900	26100	1340	6220
444	1528	1251	1041	944	993	28800	1410	6860
485	1632	1351	1145	1044	1095	31800	1460	7570
525	1727	1490	1240	1193	1217	35250	680	8400
566	Failure							

TABLE No. 20

SWITCHES
West C2-13 East C2-14

Truss No.1
Member U_1-L_1

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings C2-13	Readings C2-14	Difference C2-13	Difference C2-14	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 6	Stage 7						
0	1559	1378	0	0	0	0	0	0
81	1620	1230	61	148	44	1280	3030	251
121	1647	1158	88	220	66	1910	4470	374
162	1668	1089	109	289	90	2610	5770	511
202	1688	1028	129	350	111	3220	6950	631
242	1701	960	142	418	138	4000	8120	784
283	1703	900	144	478	167	4840	9020	969
323	1714	850	155	528	187	5430	9900	1060
364	1724	799	165	579	207	6000	10780	1180
404	1734	748	175	630	228	6610	11700	1300
444	1743	695	184	683	250	7250	12600	1420
485	1767	643	208	735	264	7660	13700	1500
525	1763	580	204	798	297	8620	14500	1690
566	Failure							

TABLE No. 21

SWITCHES
West A1-4 East A1-5

Truss No.2
Member U_1-L_1

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings A1-4	Readings A1-5	Difference A1-4 −	Difference A1-5 +	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 7	Stage 6						
0	1896	1209	0	0	0	0	0	0
81	1780	1227	116	18	49	1420	1940	278
121	1728	1232	168	23	73	2120	2770	416
162	1677	1236	219	27	96	2780	3570	545
202	1632	1240	264	31	117	3390	4280	664
242	1573	1230	323	21	151	4380	4990	860
283	1531	1230	365	21	172	4990	5600	978
323	1480	1229	416	20	198	5740	6320	1120
364	1437	1231	459	22	219	6360	6970	1250
404	1386	1229	510	20	245	7110	7690	1390
444	1339	1231	557	22	268	7770	8400	1520
485	1298	1232	598	23	288	8350	9000	1640
525	1245	1228	651	19	316	9170	9705	1800
566	Failure							

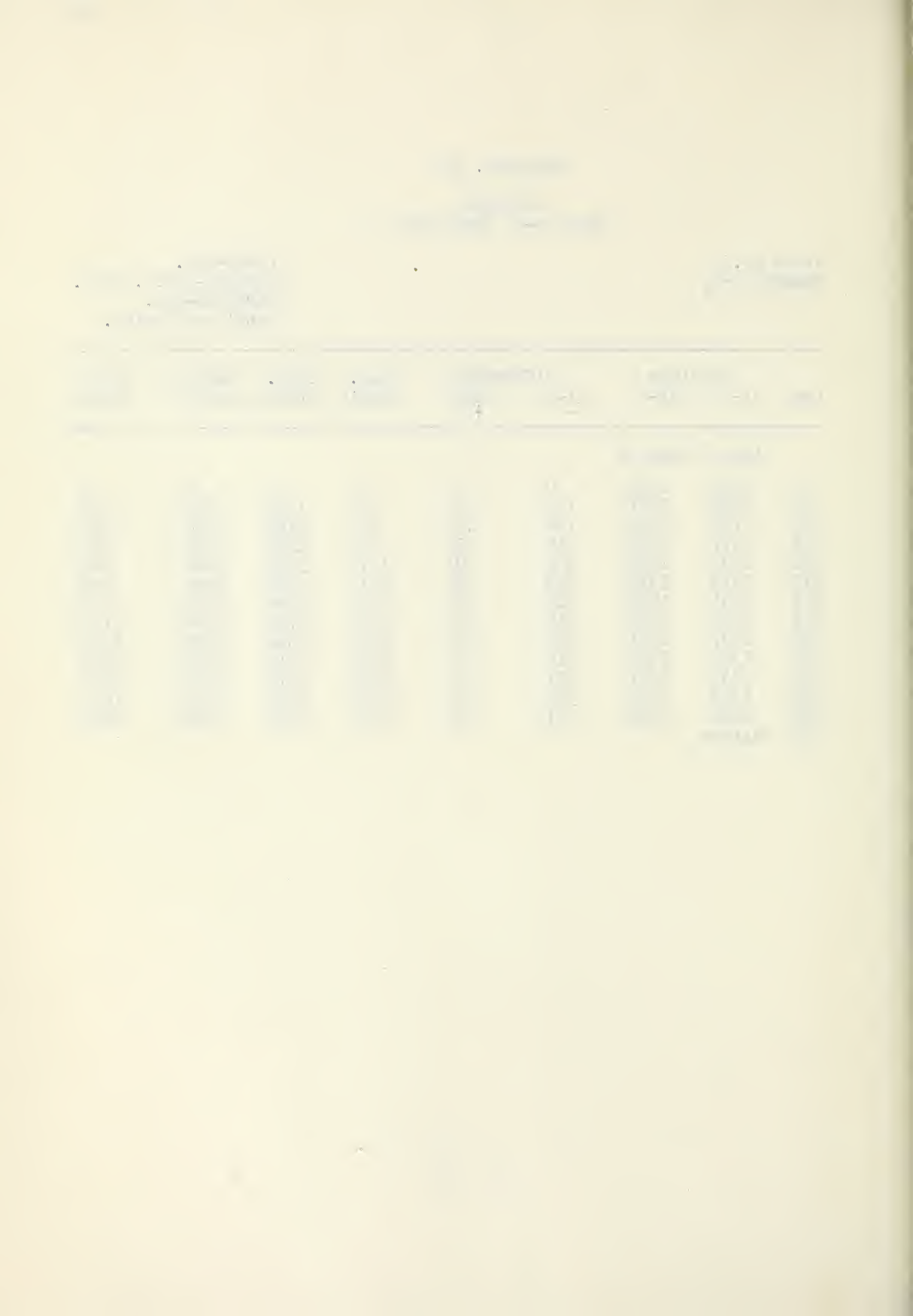


TABLE No. 22

SWITCHES
Bottom C2-15 Top C2-16

Truss No.1
Member U_1-L_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings C2-15	Readings C2-16	Difference C2-15	Difference C2-16	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 9	Stage 9						
0	261	238	0	0	0	0	0	0
81	337	329	76	91	84	2440	217	468
121	371	372	110	134	122	3540	348	680
162	410	417	149	179	164	4760	435	914
202	447	460	186	222	204	5920	521	1140
242	478	497	217	259	238	6900	609	1320
283	504	539	243	301	272	7880	840	1510
323	557	583	296	345	321	9310	710	1790
364	598	628	337	390	364	10600	768	2030
404	638	672	377	434	406	11750	825	2260
444	673	712	412	474	443	12800	900	2470
485	718	758	457	520	489	14200	913	2730
525	748	794	487	556	522	15100	1000	2900
566	Failure							

TABLE No. 23

SWITCHES
Bottom A1-6 Top A2-1

Truss No.2
Member U_1-L_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings A1-6	Readings A2-1	Difference A1-6	Difference A2-1	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 8	Stage 8						
0	498	318	0	0	0	0	0	0
81	607	362	109	44	77	2230	940	439
121	660	391	162	73	118	3320	1290	654
162	709	422	211	104	158	4580	1560	903
202	758	452	260	134	197	5720	1830	1130
242	798	474	300	156	228	6610	2090	1300
283	845	508	347	190	269	7800	2280	1540
323	881	534	383	216	300	8700	2420	1710
364	931	573	433	255	344	9970	2580	1960
404	973	608	475	290	383	11100	2680	2190
444	1011	642	513	324	419	12200	2740	2390
485	1060	682	562	364	463	13400	2870	2640
525	1098	711	600	393	497	14400	3010	2840
566	Failure							

TABLE No. 24

SWITCHES
West C2-17 East C2-18

Truss No.1
Member U_2-L_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings C2-17	Readings C2-18	Difference C2-17	Difference C2-18	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 4	Stage 5						
0	1830	1572	0	0	0	0	0	0
81	1752	1552	78	20	49	1420	825	153
121	1712	1545	118	27	73	2120	1320	229
162	1672	1533	158	39	99	2870	1730	310
202	1632	1528	198	44	121	3510	2240	379
242	1579	1505	251	67	159	4610	2670	497
283	1532	1493	298	79	189	5480	3180	593
323	1489	1484	341	88	215	6240	3670	674
364	1448	1473	382	99	241	7000	4100	756
404	1403	1464	427	108	268	7770	4630	840
444	1362	1457	468	115	292	8470	5120	915
485	1313	1445	517	127	322	9340	5650	1010
525	1261	1426	569	146	358	10400	6150	1120
566	Failure							

TABLE No. 25

SWITCHES
West A2-2 East A2-3

Truss No.2
Member U_2-L_2

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings A2-2	Readings A2-3	Difference A2-2	Difference A2-3	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 5	Stage 5						
0	1950	1521	0	0	0	0	0	0
81	1948	1422	2	99	51	1480	1410	160
121	1940	1373	10	148	79	2290	2000	248
162	1930	1325	20	196	108	3130	2550	338
202	1925	1282	25	239	132	3830	3110	414
242	1908	1227	42	294	168	4870	3660	526
283	1898	1181	52	340	196	5680	4320	614
323	1887	1130	63	391	227	6580	4750	710
364	1879	1088	71	433	252	7310	5250	790
404	1868	1038	82	483	283	8210	5800	877
444	1859	990	91	531	311	9020	6380	974
485	1853	948	97	573	335	9720	6900	1050
525	1839	887	111	634	373	10800	7600	1170
566	Failure							

TABLE No. 26

SWITCHES
Bottom Cl-l3 Top Cl-l4

Truss No.1
Member U_2-L_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings Cl-13	Readings Cl-14	Difference Cl-13	Difference Cl-14	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 7	Stage 5						
0	312	819	0	0	0	0	0	0
81	412	905	100	86	93	2700	203	292
121	468	950	156	131	144	4180	362	452
162	511	988	199	169	184	5340	435	577
202	558	1032	246	213	230	6670	478	720
242	597	1067	285	248	267	7740	535	835
283	645	1107	333	288	311	9020	652	974
323	689	1148	377	329	353	10200	695	1110
364	734	1191	422	372	397	11500	725	1240
404	781	1232	469	413	441	12800	810	1380
444	822	1272	510	453	482	14000	825	1510
485	869	1318	557	499	528	15300	840	1660
525	909	1352	597	533	565	16400	927	1770
566	Failure							

TABLE No. 27

SWITCHES
Bottom A2-4 Top A2-5

Truss No.2
Member U_2-L_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings A2-4	Readings A2-5	Difference A2-4	Difference A2-5	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 7	Stage 8						
0	138	682	0	0	0	0	0	0
81	222	758	84	76	80	2320	116	256
121	268	799	130	117	124	3600	188	396
162	305	839	167	157	162	4700	145	517
202	347	880	209	198	204	5920	159	651
242	379	917	241	235	238	6900	87	759
283	418	960	280	278	279	8090	29	890
323	451	999	313	317	315	9140	58	1050
364	498	1044	360	362	361	10500	29	1150
404	537	1089	399	407	403	11700	116	1290
444	573	1132	435	450	443	12800	217	1410
485	618	1182	480	500	490	14200	290	1560
525	651	1217	513	535	524	15200	319	1670
566	Failure							

TABLE No. 28

SWITCHES
West C1-15 East C1-16

Truss No.1
Member U_3-L_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings C1-15	Readings C1-16	Difference C1-15	Difference C1-16	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 6	Stage 5						
0	1692	1919	0	0	0	0	0	0
81	1662	1881	30	38	34	986	116	107
121	1648	1862	44	57	51	1480	189	161
162	1628	1847	64	72	68	1970	116	215
202	1611	1828	81	91	86	2500	145	272
242	1584	1800	108	119	114	3310	160	360
283	1568	1782	124	137	131	3800	189	414
323	1549	1768	143	151	147	4270	116	465
364	1532	1748	160	171	166	4820	160	526
404	1512	1729	180	190	185	5370	145	585
444	1492	1709	200	210	205	5950	145	649
485	1472	1690	220	229	225	6530	131	712
525	1440	1670	252	269	261	7570	247	825
566	Failure							

TABLE No. 29

SWITCHES
West A2-6 East B1-7

Truss No.2
Member U_3-L_3

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings A2-6	Readings B1-7	Difference A2-6	Difference B1-7	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 4	Stage 5						
0	1882	1928	0	0	0	0	0	0
81	1888	1860	6	-68	31	899	1070	98
121	1890	1830	8	-98	45	1300	1540	142
162	1888	1799	6	-129	62	1800	1960	196
202	1888	1769	6	-159	77	2230	2390	243
242	1872	1729	-10	-199	95	2760	2740	301
283	1869	1699	-13	-229	108	3130	3130	341
323	1860	1661	-22	-267	145	4210	3560	459
364	1861	1635	-21	-293	157	4550	3940	497
404	1856	1598	-26	-330	178	5170	4410	564
444	1856	1561	-26	-367	197	5710	4950	622
485	1858	1537	-24	-391	208	6040	5320	658
525	1870	1469	-12	-459	236	6840	6490	745
566	Failure							

TABLE No. 30

SWITCHES
Bottom B2-8 Top B2-9

Truss No.1
Member U_3-L_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-pounds

Load	Readings B2-8	Readings B2-9	Difference B2-8	Difference B2-9	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 7	Stage 7						
0	259	639	0	0	0	0	0	0
81	308	658	49	19	34	986	435	104
121	331	669	72	30	51	1480	610	158
162	349	680	90	41	66	1910	710	204
202	371	693	112	54	83	2410	840	256
242	384	698	125	59	92	2670	955	286
283	409	709	150	70	110	3190	1160	342
323	428	721	169	82	126	3660	1260	392
364	451	739	192	100	146	4230	1340	453
404	469	752	210	113	162	4700	1410	503
444	490	769	231	130	181	5250	1470	561
485	508	796	249	157	203	5890	1340	630
525	520	793	261	154	208	6040	1550	646
566	Failure							

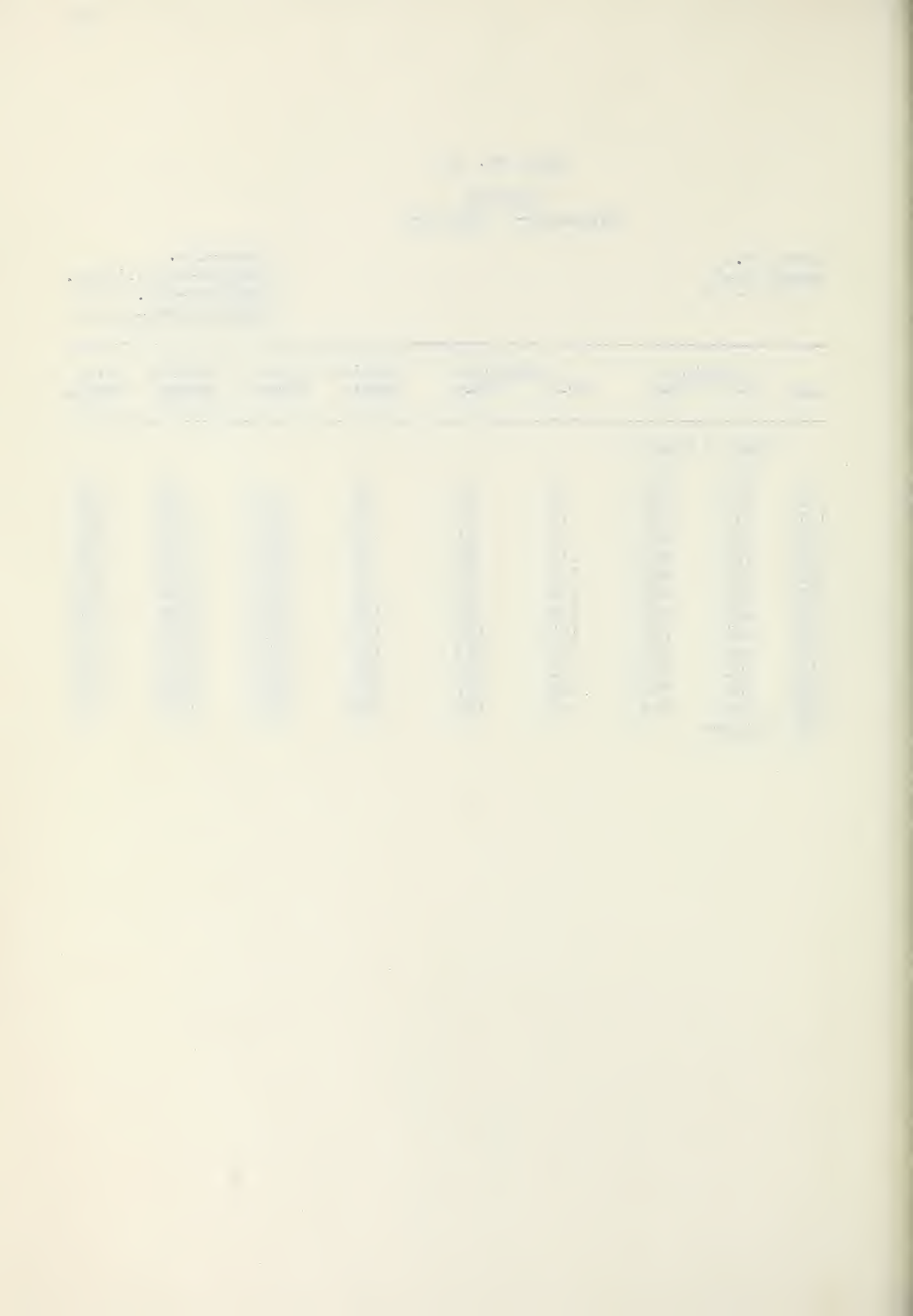


TABLE No. 31

SWITCHES
Bottom Bl-8 Top Bl-9

Truss No.2
Member U_3-L_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings Bl-8	Readings Bl-9	Difference Bl-8	Difference Bl-9	Tension Strain	Tension Stress	Bending Stress	Total Stress
	Stage 6	Stage 6						
0	409	3	0	0	0	0	0	0
81	442	34	33	31	32	928	29	102
121	458	51	49	48	49	1420	14	156
162	470	62	61	59	60	1740	29	191
202	488	80	79	77	78	2260	29	248
242	497	88	88	85	87	2520	43	277
283	514	103	105	100	103	2980	73	328
323	523	112	114	109	112	3250	73	358
364	546	131	137	128	133	3860	130	424
404	562	148	153	145	149	4320	116	476
444	578	164	169	161	165	4780	116	526
485	605	192	196	189	193	5590	101	615
525	614	198	205	195	200	5800	145	638
566	Failure							

TABLE No. 32

SWITCHES
West B1-12 East B2-7

Truss No.1
Member U_4-L_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings B1-12	Readings B2-7	Difference B1-12	Difference B2-7	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 6	Stage 5						
0	1419	1373	0	0	0	0	0	0
81	1389	1367	30	6	18	522	349	57
121	1378	1360	41	13	27	783	406	85
162	1360	1350	59	23	41	1090	522	119
202	1348	1345	71	28	50	1450	624	158
242	1328	1328	91	45	68	1970	667	215
283	1314	1315	105	58	82	2380	681	260
323	1301	1306	118	67	93	2700	740	294
364	1290	1299	129	74	102	2960	797	323
404	1277	1290	142	83	113	3480	855	379
444	1262	1280	157	93	125	3630	928	396
485	1253	1271	166	102	134	3890	928	424
525	1242	1248	177	125	151	4380	754	478
566	Failure							

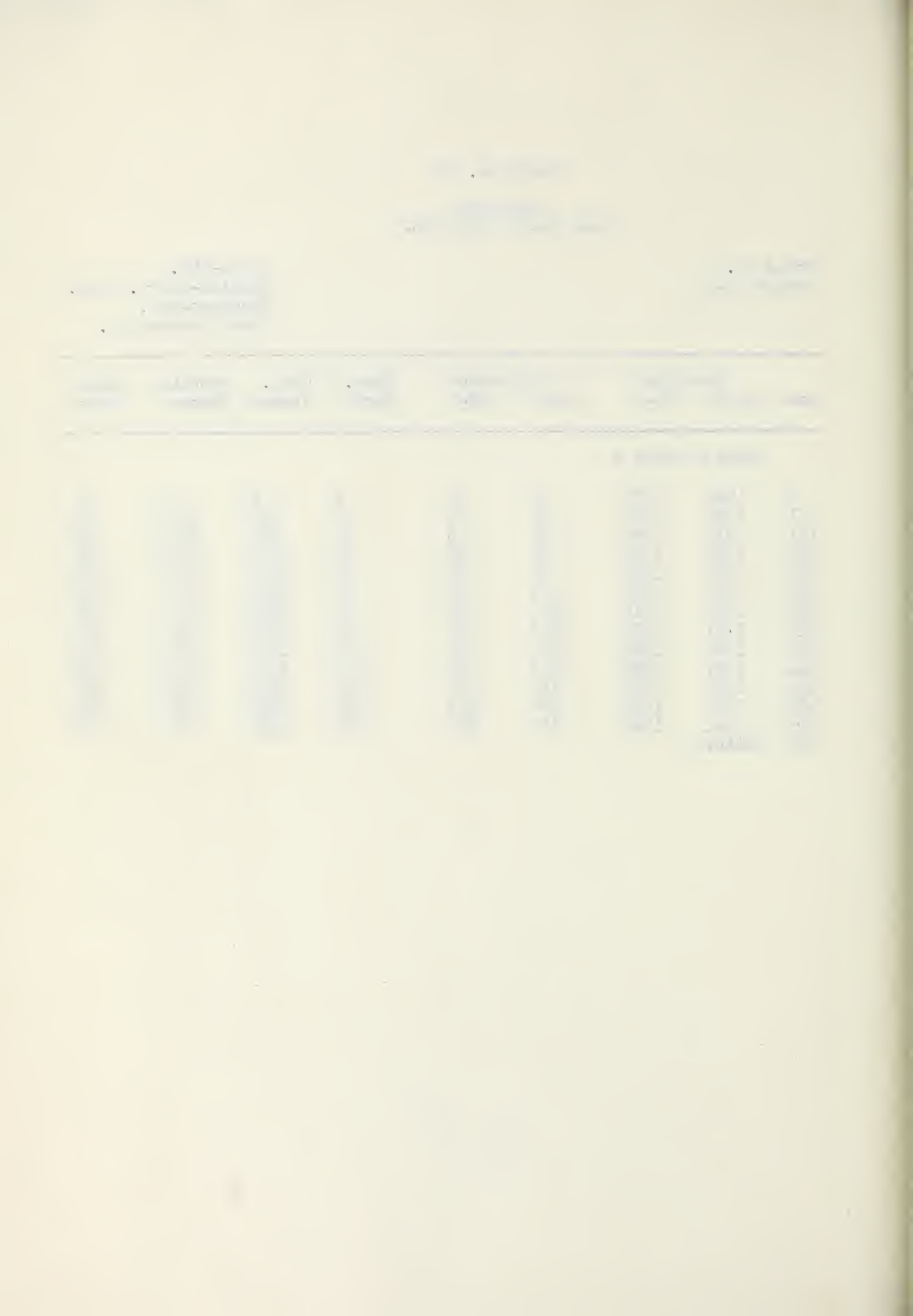


TABLE No. 33

SWITCHES
West Bl-10 East Bl-11

Truss No.2
Member U_4-L_4

Loads-lbs.
Strains-micro.in/in.
Stresses-psi.
Total stress-lbs.

Load	Readings Bl-10	Readings Bl-11	Difference Bl-10	Difference Bl-11	Comp. Strain	Comp. Stress	Bending Stress	Total Stress
	Stage 4	Stage 6						
0	1718	1202	0	0	0	0	0	0
81	1698	1190	20	12	16	474	116	52
121	1688	1183	30	19	25	725	160	80
162	1673	1173	45	29	37	1070	232	118
202	1660	1169	58	33	46	1330	363	146
242	1638	1151	80	51	66	1910	421	210
283	1629	1149	89	53	71	2060	520	226
323	1611	1137	107	65	86	2500	610	275
364	1602	1134	116	68	92	2670	695	294
404	1589	1123	129	79	104	3020	725	332
444	1581	1112	137	90	114	3310	380	360
485	1574	1111	144	91	118	3420	769	376
525	1552	1087	166	115	141	4080	740	448
566	Failure							

TABLE No. 34

DEFLECTIONS IN INCHES

Load	Readings			Deflections		
	No.2 Ruler	No.1 Ames Dial	No.1 Ruler	No.2 Ruler	No.1 Ames Dial	No.1 Ruler
0	8.40		7.10	0	0	0
81	8.39	9.999	7.09	0.01		0.01
			Reset		Correct to	
81			7.03		0.01	
121	8.34	9.937	6.97	0.06	0.07	0.06
162	8.28	9.880	6.92	0.12	0.13	0.12
202	8.23	9.824	6.87	0.17	0.185	0.17
242	8.17	9.772	6.82	0.23	0.24	0.22
283		9.722	6.78		0.29	0.26
323			6.72			0.32
		Reset	Reset		Correct to	
323		9.939	6.74		0.32	
364		9.890	6.69		0.37	0.37
404			6.62			0.42
444			6.57			0.47
			From Floor			
444			7.45			
485			7.53			0.55
525			7.58			0.60
566	Failure					

TABLE No. 35

TENSION TEST

Truss No.1
Member U_0-L_1

Strains-micro.in./in.
Load-lbs.
Unit Stress-psi.

Load	Readings Bottom	Readings Top	Difference Bottom	Difference Top	Mean Strain	Unit Stress
	Stage 5	Stage 6				
0(20)	40	452	0	0	0	82
525	123	518	83	66	75	2150
790	158	557	118	107	113	3230
1210	216	621	176	179	178	4990
1520	259	666	219	214	217	6220
1970	319	733	279	281	280	8060
3050	463	899	423	447	435	12500
3950	583	1047	543	595	569	16200
5075	732	1203	692	751	722	20800
5980	854	1340	814	888	851	24500
6950	982	1483	942	1031	987	28450
7930	1118	1623	1078	1171	1125	32450
8960	1268	1765	1228	1313	1271	36700
9880	1401	1891	1361	1439	1400	40400
		Stage 7				
9880		932				
10600	Yield Point					43400
14675	Ultimate					60000

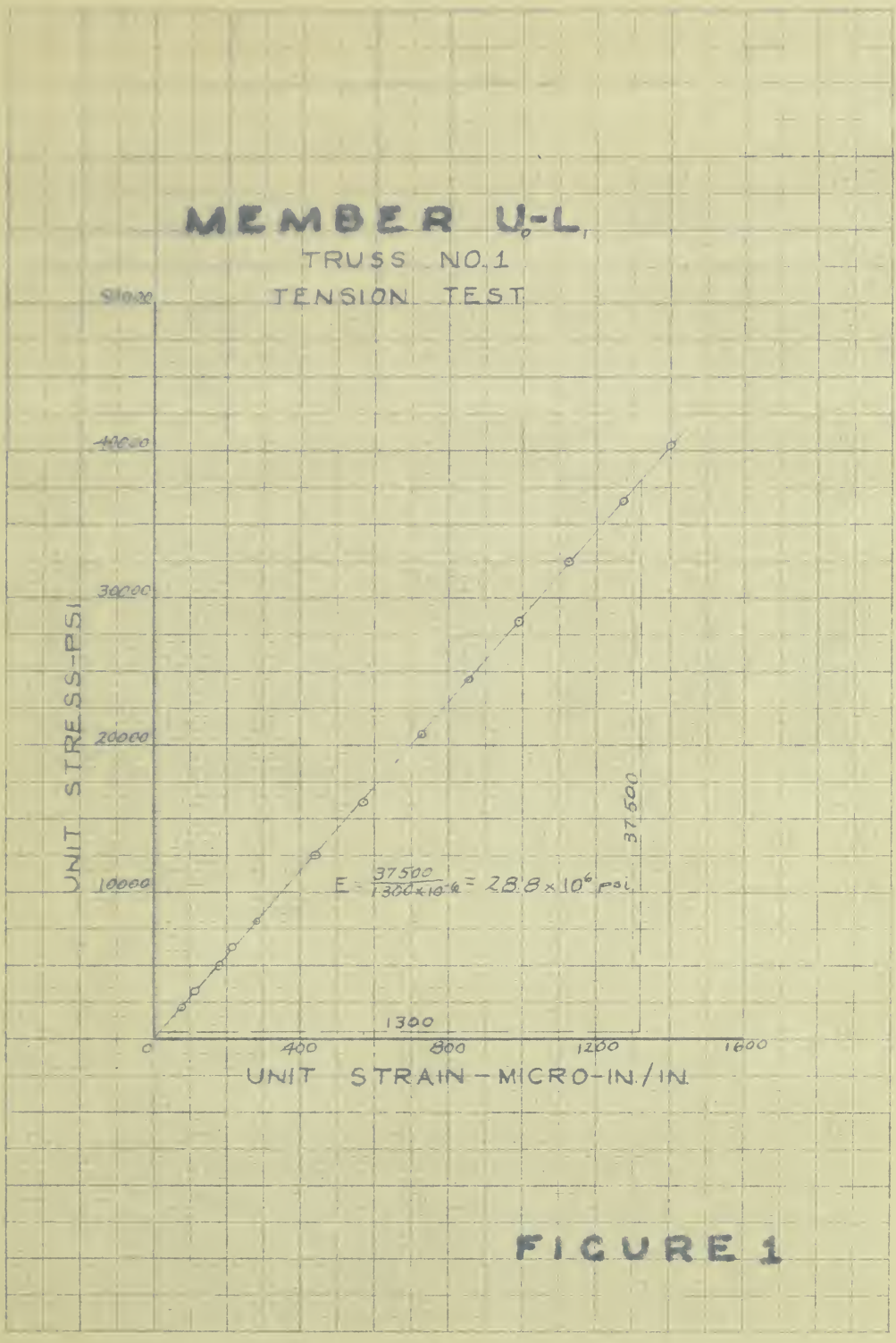

MEMBER U_0-L_1
TRUSS NO. 1
TENSION TEST
40000
30000
20000
10000
UNIT STRESS-PSI
37500
$E = \frac{37500}{1300 \times 10^{-6}} = 28.8 \times 10^6$ psi
1300
0
400
800
1200
1600
UNIT STRAIN - MICRO-IN./IN.

FIGURE 1

TABLE No. 36

TENSION TEST

Truss No.1
Member L_3-L_4

Strains-micro.in/in.
Load-lbs.
Unit Stress-psi.

Load	Readings Bottom	Readings Top	Difference Bottom	Difference Top	Mean Strain	Unit Stress
	Stage 5	Stage 6				
0(50)	915	101	0	0	0	204
1200	1104	242	189	141	165	4900
2250	1271	371	356	270	313	9190
3150	1412	479	497	378	438	12800
	Stage 6					
3150	442					
4150	598	603	653	502	578	17000
5050	732	709	787	608	698	20600
6100	892	837	947	736	842	24900
7050	1042	948	1097	847	972	28800
8025	1197	1057	1252	956	1104	32800
9050	1361	1170	1416	1069	1243	36900
9950	Yield Point					40600
14400	Ultimate					58800

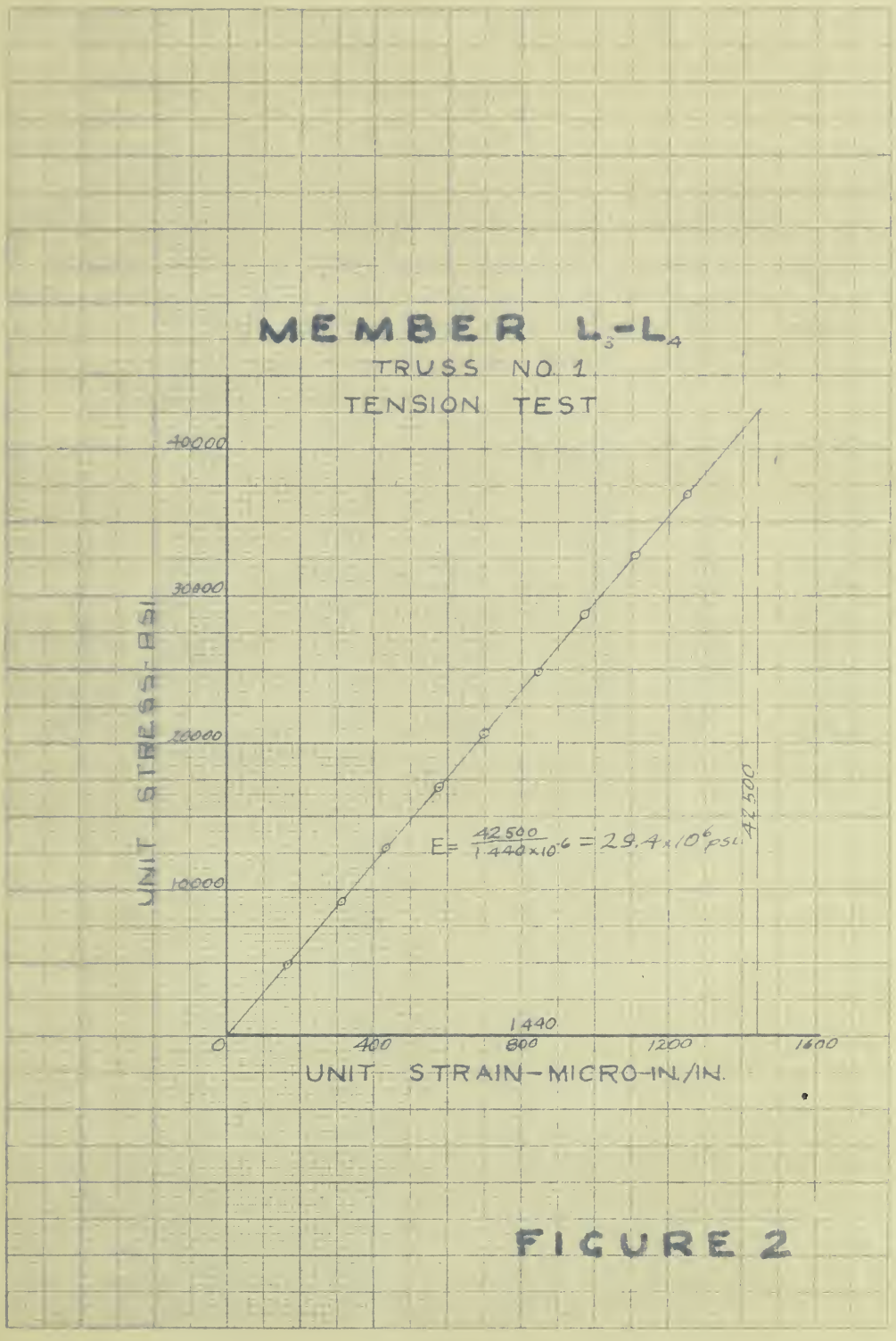

MEMBER L_3-L_4
TRUSS NO. 1
TENSION TEST
40000
30000
20000
10000
UNIT STRESS-PSI
42,500
$E = \frac{42500}{1440 \times 10^{-6}} = 29.4 \times 10^6$ psi.
0
400
800
1440
1200
1600
UNIT STRAIN-MICRO-IN./IN.

FIGURE 2

TABLE No. 37

TENSION TEST

Truss No.1
Member U_3-L_4

Strains-micro.in/in.
Load-lbs.
Unit Stress-psi.

Load	Readings Bottom	Readings Top	Difference Bottom	Difference Top	Mean Strain	Unit Stress
	Stage 7	Stage 7				
0(30)	238	444	0	0	0	278
495	389	582	151	138	145	4630
780	484	671	246	227	237	7290
1180	614	790	376	346	361	11000
1620	759	923	521	479	500	15100
1990	878	1032	640	588	614	18600
2370	998	1146	760	702	731	22200
2970	1188	1323	950	879	915	27800
3490	1352	1479	1114	1035	1075	32600
3900	1479	1604	1241	1160	1201	36400
4090	1538	1663	1300	1219	1260	38200
4540	1678	1799	1440	1355	1398	42400
5000	1822	1932	1584	1488	1536	46700
6650	Yield Point					62200
8580	Ultimate					81100

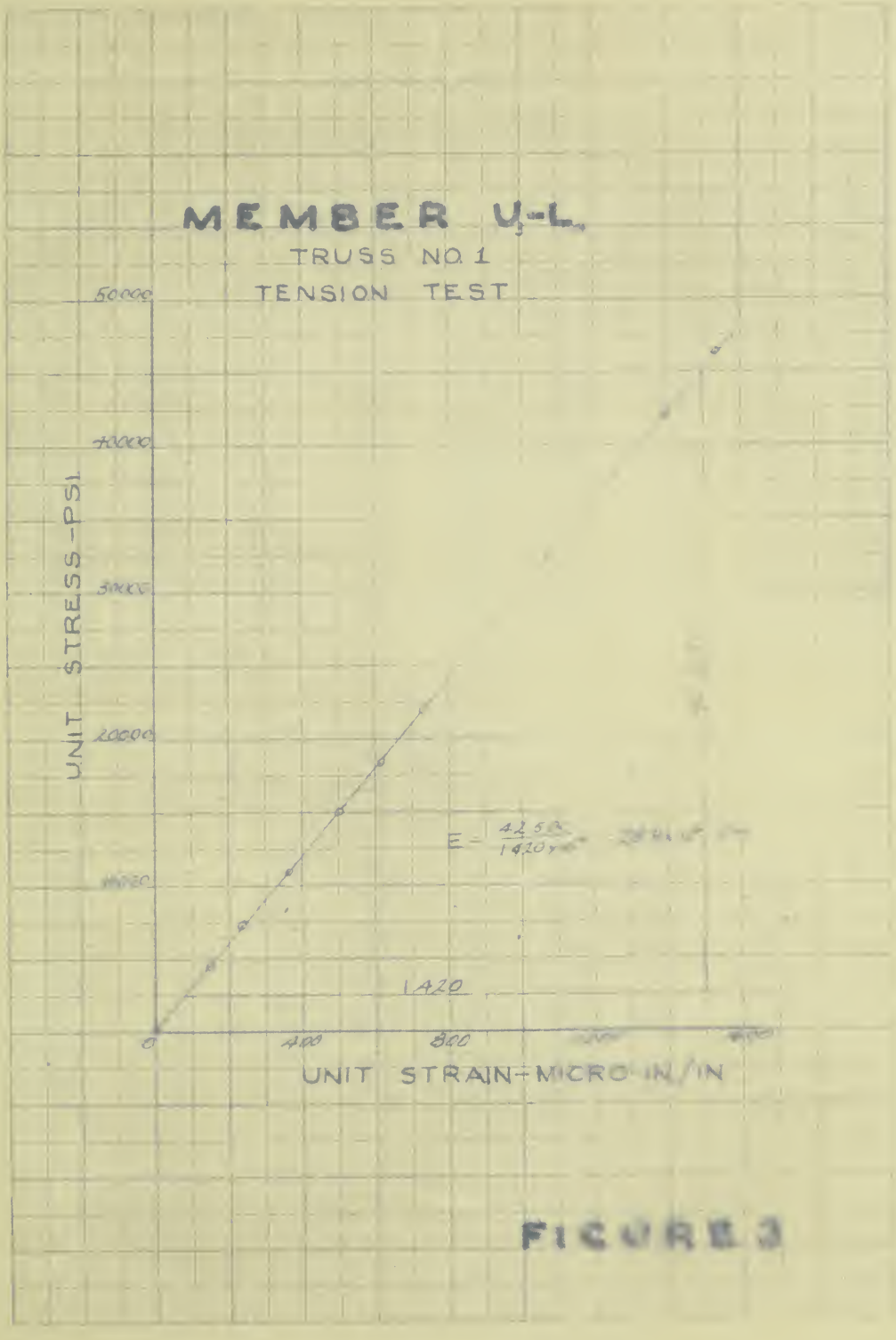

MEMBER U₂-L₂
TRUSS NO. 1
TENSION TEST
UNIT STRESS-PSI
50000
40000
30000
20000
10000
E = 4250 / 1420
1420
0
400
800
UNIT STRAIN-MICRO IN./IN

FIGURE 3

TABLE No. 38

ACTUAL COMPUTED TOTAL STRESSES IN POUNDS

Truss No.1 Loads-lbs/panel point

Load	$P_{U_0-L_1}$	$P_{L_1-L_2}$	$P_{U_1-L_2}$	$P_{L_2-L_3}$	$P_{U_2-L_3}$	$P_{L_3-L_4}$	$P_{U_3-L_4}$	$P_{U_1-L_1}$
81	676	611	499	1070	294	1340	100	284
121	1010	914	745	1600	439	2010	150	424
162	1350	1220	998	2130	588	2690	202	567
202	1690	1530	1240	2660	734	3350	250	707
242	2020	1830	1490	3190	879	4010	301	847
283	2360	2140	1740	3730	1030	4690	351	991
323	2700	2440	1990	4260	1170	5360	400	1130
364	3040	2750	2240	4800	1320	6040	452	1270
404	3380	3050	2490	5330	1470	6700	501	1410
444	3710	3350	2730	5850	1610	7360	552	1550
485	4050	3660	2990	6400	1760	8050	602	1700
525	4380	3960	3240	6920	1910	8710	652	1840

Load	$P_{U_2-L_2}$	$P_{U_3-L_3}$	$P_{U_4-L_4}$	$P_{U_0-U_1}$	$P_{U_1-U_2}$	$P_{U_2-U_3}$	$P_{U_3-U_4}$
81	202	121	81	611	1070	1340	1440
121	302	182	121	914	1600	2010	2150
162	405	243	162	1220	2130	2690	2870
202	505	303	202	1530	2670	3350	3580
242	605	363	242	1830	3190	4010	4290
283	707	424	283	2140	3730	4690	5010
323	808	484	323	2440	4260	5360	5730
364	910	546	364	2750	4800	6040	6460
404	1010	606	404	3050	5330	6700	7180
444	1110	665	444	3350	5850	7360	7880
485	1210	727	485	3660	6400	8050	8600
525	1310	787	525	3960	6920	8710	9310

TABLE No. 39

ACTUAL COMPUTED TOTAL STRESSES IN POUNDS

Truss No.2 Load-lbs/panel point

Load	$P_{U_0-L_1}$	$P_{L_1-L_2}$	$P_{U_1-L_2}$	$P_{L_2-L_3}$	$P_{U_2-L_3}$	$P_{L_3-L_4}$	$P_{U_3-L_4}$	$P_{U_1-L_1}$
81	689	618	521	1100	299	1370	101	284
121	1030	925	778	1640	446	2050	151	424
162	1380	1240	1040	2190	598	2740	202	567
202	1720	1540	1300	2740	745	3420	252	707
242	2060	1850	1560	3280	894	4090	302	847
283	2410	2160	1820	3840	1040	4790	354	990
323	2740	2460	2080	4370	1190	5460	405	1130
364	3090	2780	2340	4930	1340	6160	455	1280
404	3440	3090	2600	5470	1490	6850	506	1420
444	3780	3390	2860	6010	1640	7510	556	1560
485	4120	3710	3120	6570	1790	8210	607	1700
525	4460	4010	3380	7110	1940	8890	657	1840

Load	$P_{U_2-L_2}$	$P_{U_3-L_3}$	$P_{U_4-L_4}$	$P_{U_0-U_1}$	$P_{U_1-U_2}$	$P_{U_2-U_3}$	$P_{U_3-U_4}$
81	202	121	81	618	1100	1370	1460
121	302	182	121	925	1640	2050	2180
162	405	243	162	1240	2190	2740	2920
202	505	303	202	1540	2740	3420	3640
242	605	363	242	1850	3280	4090	4370
283	707	424	283	2160	3840	4790	5110
323	807	484	323	2460	4370	5460	5830
364	910	546	364	2780	4830	6160	6560
404	1010	606	404	3090	5470	6850	7300
444	1110	666	444	3390	6010	7510	8010
485	1210	728	485	3710	6570	8210	8750
525	1310	787	525	4010	7110	8890	9490

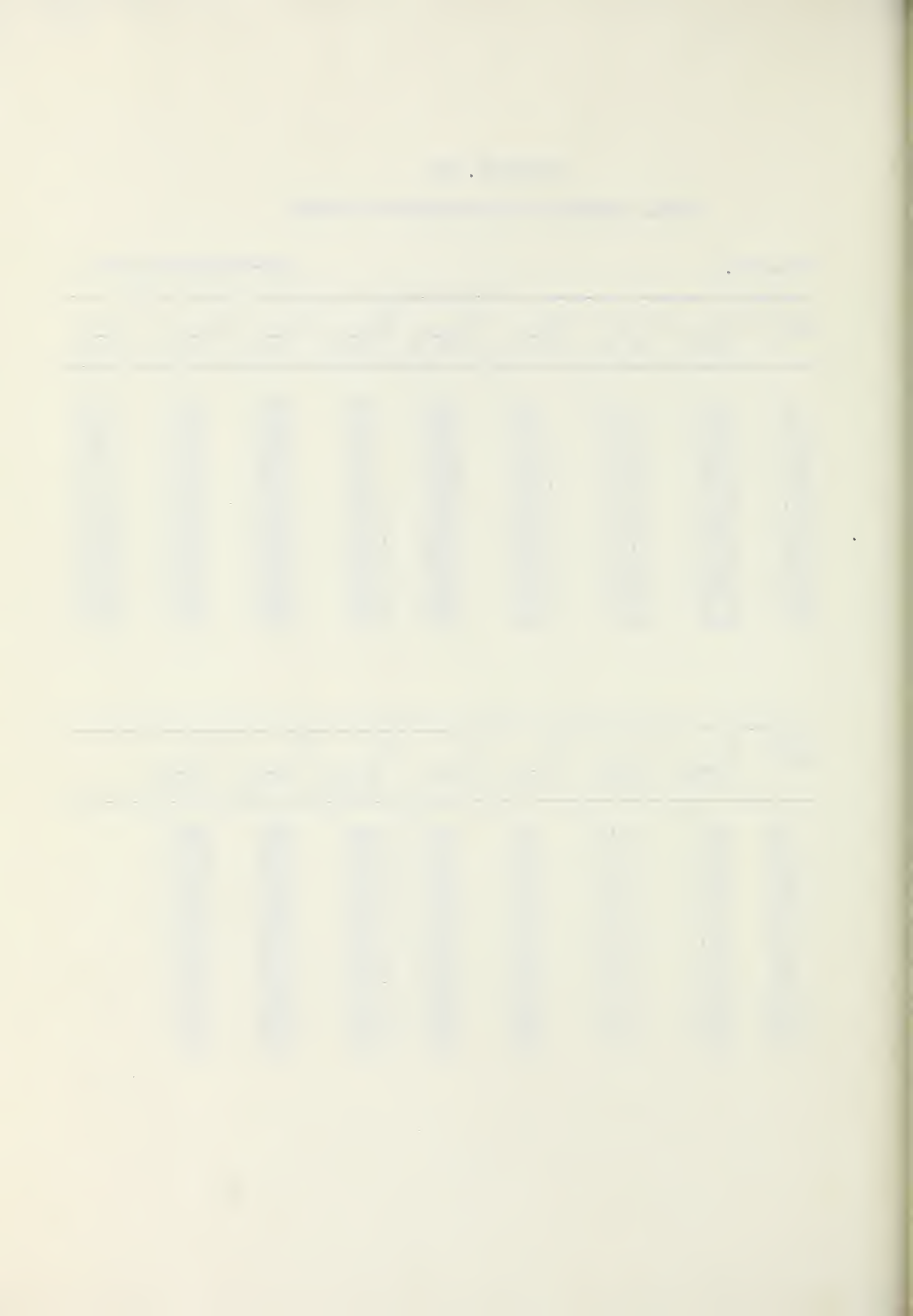


TABLE No. 40

MAXIMUM UNIT STRESSES IN PSI

Upper Chord — Load-lbs/panel point

Load	U_0-U_1 #1		U_0-U_1 #2		U_1-U_2 #1		U_1-U_2 #2	
	Computed	Observed	Computed	Observed	Computed	Observed	Computed	Observed
81	2400	4030	2420	1570	3150	1910	3140	2120
121	3580	5460	3620	2230	4720	2870	4700	2910
162	4790	6670	4840	2850	6300	3890	6290	3700
202	5970	8040	6040	3350	7720	5100	7860	4650
242	7160	9420	7220	4550	9410	6600	9410	6190
283	8380	10600	8460	4970	11000	7840	11000	7290
323	9560	11200	9630	5490	12600	8770	12600	7970
364	10800	12200	10900	6230	14200	9920	14000	9070
404	12000	13100	12100	6890	15700	10800	15700	9980
444	13100	13800	13300	7430	17300	11600	17300	10900
485	14400	14400	14500	8120	18900	12500	18900	11800
525	15500	15500	15700	8850	20400	13700	20400	12900

Load	U_2-U_3 #1		U_2-U_3 #2		U_3 -U_4 #1		U_3-U_4 #2	
	Computed	Observed	Computed	Observed	Computed	Observed	Computed	Observed
81	3590	2830	3430	2800	3690	2410	3770	2410
121	5360	4270	5130	3990	5500	3490	5630	3730
162	7170	5540	6860	4710	7380	4770	7540	5030
202	8800	7160	8560	6070	9050	6390	9390	6340
242	10700	8710	10300	7610	11000	7580	11300	7930
283	12500	10100	12000	9070	12900	8950	13200	9220
323	14300	11200	13700	10100	14700	10000	15100	10300
364	16100	12800	15400	11400	16600	11500	16900	11700
404	17900	14100	17200	12800	18400	12700	18900	12800
444	19700	14700	18800	13900	20200	13700	20700	14100
485	21500	16400	20500	15200	22100	14900	22600	15700
525	23300	17700	22300	16000	23900	16100	24400	16500

TABLE No. 41

MAXIMUM UNIT STRESSES IN PSI

Lower Chord — Load-lbs/panel point

Load	U_0-U_1 #1		U_0-L_1 #2		L_1-L_2 #2		L_1-L_2 #2	
	Computed	Observed	Computed	Observed	Computed	Observed	Computed	Observed
81	2760	5520	2830	3380	2460	4360	2530	4820
121	4240	7830	4240	5000	3670	6340	3780	7060
162	5520	9860	5680	6660	4900	8080	5070	9130
202	6920	11900	7070	8280	6120	9620	6300	10700
242	8260	13300	8470	9750	7350	10900	7570	12200
283	9660	15700	9900	11100	8570	12400	8830	14200
323	11100	16900	11300	13000	9800	13900	10100	15700
364	12400	18300	12700	14300	11000	15100	11400	17100
404	13800	19800	14200	16000	12300	16500	12600	18800
444	15200	21500	15600	17800	13500	18000	13900	20300
485	16600	23200	16900	19100	14700	19500	15200	22000
525	18000	24700	18400	20500	15900	20800	16400	23500

Load	L_2-L_3 #1		L_2-L_3 #2		L_3-L_4 #2		L_3-L_4 #2	
	Computed	Observed	Computed	Observed	Computed	Observed	Computed	Observed
81	4270	5000	4600	4890	5380	7530	5750	5880
121	6380	7520	6890	7310	8040	11300	8600	8800
162	8510	9910	9200	9760	10700	14400	11500	11700
202	10600	12100	11500	11800	13400	17400	14400	14100
242	12800	14200	13800	13800	16000	20100	17200	16600
283	14900	16300	16100	16000	18800	23000	20200	19200
323	17000	18500	18300	18400	21400	25800	22900	22000
364	19200	20500	20700	20600	24200	28700	25800	24700
404	21300	22600	23000	22900	26800	31400	28800	27400
444	23400	24900	25200	25200	29400	34300	31500	30200
485	25600	27000	27600	27800	32200	38200	34500	33300
525	27700	29100	29900	29000	34800	41000	37300	35900

TABLE No. 42

MAXIMUM UNIT STRESSES IN PSI

VERTICALS

Load-lbs/panel point

Load	U_1-L_1 #1 Computed	Observed	U_1-L_1 #2 Computed	Observed	U_2-L_2 #1 Computed	Obser.	U_2-L_2 #2 Computed	Observed
81	1450	4310		3360	1870	2250		2890
121	2160	6380		4890	2800	3440		4290
162	2900	8380		6350	3750	4600		5680
202	3600	10200		7670	4680	5700		6940
242	4320	12100		9370	5600	7280		8530
283	5060	13900		10600	6550	8660		10000
323	5760	15300		12100	7480	9910		11300
364	6490	16800		13300	8430	11100		12600
404	7200	18300	Same as No. 1	14800	9350	12400	Same as No. 1	14000
444	7910	19800		16200	10300	13600		15400
485	8670	21300		17400	11200	15000		16600
525	9390	23100		18900	12100	16600		18400

Load	U_3-L_3 #1 Computed	Observed	U_3-L_3 #2 Computed	Observed	U_4-L_4 #1 Computed	Obser.	U_4-L_4 #2 Computed	Observed
81	1110	1100		1970	770	871	737	590
121	1670	1670		2840	1110	1190	1100	885
162	2230	2090		3760	1490	1610	1470	1300
202	2780	2640		4620	1850	2070	1840	1690
242	3330	3470		5500	2220	2640	2200	2330
283	3890	3990		6260	2600	3060	2580	2580
323	4440	4390		7770	2960	3440	2940	3110
364	5010	4980		8490	3340	3760	3310	3370
404	5560	5520	Same as No. 1	9580	3710	4330	3680	3750
444	6110	6100		10700	4070	4550	4040	3990
485	6680	6660		11400	4450	4810	4410	4190
525	7220	7820		13300	4810	5130	4770	4820

TABLE No. 43

MAXIMUM UNIT STRESSES IN PSI

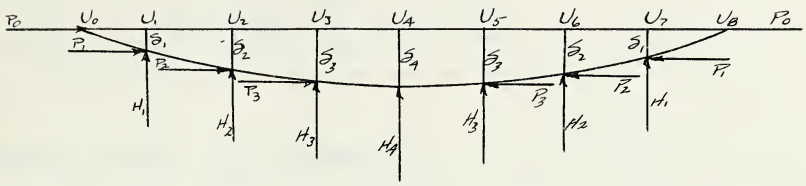
DIAGONALS

Load-lbs/panel point

	U_1-L_2 #1		U_1-L_2 #2		U_2-L_3 #1		U_2-L_3 #2	
Load	Computed	Observed	Computed	Observed	Computed	Observed	Computed	Observed
81	2600	2660	2640	3170	2720	2900		2440
121	3880	3890	3950	4610	4070	4540		3790
162	5200	5200	5280	6140	5450	5780		4850
202	6460	6440	6600	7550	6800	7150		6080
242	7760	7510	7920	8700	8140	8280	Same as No. 1	6990
283	9060	8720	9240	10100	9540	9670		8120
323	10400	10000	10600	11100	10800	10900		9200
364	11700	11300	11900	12600	12200	12200		10500
404	13000	12600	13200	13800	13600	13600		11800
444	14300	13700	14500	14900	14900	14800		13100
485	15600	15100	15800	16300	16300	16200		14500
525	16900	16100	17200	17400	17700	17300		15500

	U_3-L_4 #1		U_3-L_4 #2	
Load	Computed	Observed	Computed	Observed
81	935	1420	917	957
121	1400	2090	1370	1430
162	1890	2620	1840	1770
202	2340	3250	2290	2290
242	2810	3630	2740	2560
283	3280	4350	3220	3050
323	3740	4920	3680	3320
364	4230	5570	4140	3990
404	4680	6110	4600	4440
444	5110	6720	5050	4900
485	5630	7230	5510	5690
525	6090	7590	5970	5950

Buckling of Top Chord



Plan of Top Chord in Buckled Position

Assumptions: Top chord of constant cross-section
Ends fixed against lateral movement
H_1, H_2, etc., are the values of the horizontal elastic forces.
Let values of H_1, H_2, H_3, H_4 be given by equations of the form $H_1 = C_1 \delta_1$, etc.

The C's are the elastic coefficients and are dependent upon h (depth of truss) and the moments of inertia of the verticals and diagonals.

The support of a cantilever type column:

$$\delta_1 = \frac{H_1 h^3}{3E I_1} \quad \text{(Cantilever Deflection Formula)}$$

$$C_1 = \frac{H_1}{\delta_1} = \frac{3E I_1}{h^3} \text{, etc.}$$

$$y = \delta_4 \sin \frac{\pi x}{\ell} \quad \text{(i.e. sine curve deformation)}$$

Then $\delta_1 = \delta_4 \sin \frac{\pi}{8}$; $\delta_2 = \delta_4 \sin \frac{\pi}{4}$; $\delta_3 = \delta_4 \sin \frac{3\pi}{8}$

$$\frac{dy}{dx} = \frac{\pi}{\ell} \delta_4 \cos \frac{\pi x}{\ell}; \quad \frac{d^2y}{dx^2} = -\frac{\pi^2}{\ell^2} \delta_4 \sin \frac{\pi x}{\ell}$$

The work done by each H is given by:

$$W_L = \tfrac{1}{2} H_1 \delta_1, \text{ etc.}$$

$$H_1 = \frac{3EI_1}{h^3} \delta_1 = \frac{3EI_1}{h^3} \delta_4 \sin \frac{\pi}{8}$$

$$W_{L1} = \frac{3}{2} \frac{EI_1}{h^3} \delta_4^2 \sin^2 \frac{\pi}{8}$$

Therefore, total W_L is given by:

$$W_{TL} = 2 \cdot \frac{3}{2} \cdot \frac{EI_1}{h^3} \delta_4^2 \; 0.147 + 2 \cdot \frac{3}{2} \cdot \frac{EI_2}{h^3} \delta_4^2 \; 0.500$$

$$+ 2 \cdot \frac{3}{2} \cdot \frac{EI_3}{h^3} \delta_4^2 \; 0.852 + \frac{3}{2} \frac{EI_4}{h^3} \delta_4^2$$

Therefore, $W_{TL} = \delta_4^2 \left\{ \frac{3EI_1}{h^3} 0.147 + \frac{3EI_2}{h^3} \cdot \frac{1}{2} + \frac{3EI_3}{h^3} 0.852 + \frac{3}{2} \frac{EI_4}{h^3} \right\}$

But, $I_2 = I_3 = I_4$

Hence: $W_{TL} = 0.441 \frac{EI_1}{h^3} \delta_4^2 + \frac{EI_2}{h^3} \delta_4^2 \left\{ 1.5 + 2.556 + 1.5 \right\}$

$$= 0.441 \frac{EI_1}{h^3} \delta_4^2 + \frac{5.56 \, EI_2}{h^3} \delta_4^2$$

If P_0, P_1, P_2, P_3 represent the horizontal components of the diagonsls, the work done by these forces may be arrived at as follows:

Since $\frac{dy}{dx}$ is small,

$$ds - dx = dx \left\{ \sqrt{1 + \left(\frac{dy}{dx}\right)^2} - 1 \right\} = \left[\left\{ 1 + \tfrac{1}{2} \left(\frac{dy}{dx}\right)^2 + \text{--} \right\} - 1 \right] dx$$

$$= \tfrac{1}{2}\left(\frac{dy}{dx}\right)^2 dx$$

Therefore, work $= \frac{P}{2}\int_0^{\ell}\left(\frac{dy}{dx}\right)^2 dx$ where the limits represent the distance between the pairs of loads.

Thus, the work done by the components, W_{TC}, is given by:

$$W_{TC} = \frac{P_0}{2}\int_0^{\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx + \frac{P_1}{2}\int_0^{3/4\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx$$

$$+ \frac{P_2}{2}\int_0^{\frac{1}{2}\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx + \frac{P_3}{2}\int_0^{\frac{1}{4}\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx$$

Assume the same load at each panel point and the stress in $U_3 - U_5 = P$

$$P_0 = \frac{7}{16}P;\quad P_1 = \frac{5}{16}P;\quad P_2 = \frac{3}{16}P;\quad P_3 = \frac{1}{16}P$$

$$W_{TC} = \frac{7}{32}P\int_0^{\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx + \frac{5}{32}P\int_0^{3/4\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx$$

$$+ \frac{3}{32}P\int_0^{\frac{1}{2}\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx + \frac{1}{32}P\int_0^{\frac{1}{4}\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx$$

$$= \frac{P\pi^2}{\ell^2}\delta_4^2\left\{\frac{7}{32}\int_0^{\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx + \frac{5}{32}\int_0^{3/4\ell}\delta_4^2\frac{\pi^2}{\ell^2}\cos^2\right.$$

$$\frac{\pi x}{\ell}\,dx \;+\; \frac{3}{32}\int_0^{\frac{1}{2}\ell} \delta_4^2\,\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx \;+\; \frac{1}{32}\int_0^{\frac{1}{4}\ell} \delta_4^2\,\frac{\pi^2}{\ell^2}\cos^2\frac{\pi x}{\ell}\,dx\Bigg\}$$

$$W_{TC} = \frac{\pi^2 P}{\ell^2}\,\delta_4^2\left\{\frac{\ell}{2}\left(0.219 + 0.0924 + 0.0234 + 0.0128\right)\right\}$$

$$W_{TC} = \underline{0.347\,\frac{\pi^2 P}{2\ell}\,\delta_4^2}$$

The internal work due to bending is given by:

$$U = \tfrac{1}{2} E I \int_0^{\ell} \left(\frac{d^2y}{dx^2}\right)^2 dx$$

$$= \tfrac{1}{2} E I \int_0^{\ell} \frac{\pi^4}{\ell^4}\,\delta_4^2 \sin^2\frac{\pi x}{\ell}\,dx$$

$$= \frac{\pi^4 EI}{2\ell^4}\,\delta_4^2 \int_0^{\ell} \sin^2\frac{\pi x}{\ell}\,dx$$

$$= \frac{\pi^4 EI}{2\ell^4}\,\delta_4^2 \left[\frac{1}{2}\cdot\frac{\ell}{4}\left(\frac{\pi x}{\ell} - \sin\frac{\pi x}{\ell}\cos\frac{\pi x}{\ell}\right)\right]_0^{\ell}$$

$$= \underline{\frac{\pi^4 EI\,\delta_4^2}{2\ell^4}\cdot\frac{\ell}{2}}$$

Equating the work,

$$0.347 \frac{\pi^2 P}{2\ell} \delta_4^2 = \frac{\pi^4 EI}{4\ell^3} \delta_4^2 + 0.441 \frac{EI_1}{h^3} \delta_4^2 + 5.56 \frac{EI_2}{h^3} \delta_4^2$$

$$P_{cr} = \frac{2\ell E}{0.347\pi^2} \left[\frac{\pi^4 I}{4\ell^3} + \frac{0.441 I_1 + 5.556 I_2}{h^3} \right]$$

From Data:

$E = 29 \times 10^6$ psi.

$= 16' - 0" = 192"$

$h = 10\ 5/8"$

$I_1 = 0.00305^{in.^4}$

$I_2 = 0.000945^{in.^4}$

$I = 0.220^{in.^4}$

$$\underline{\underline{P_{cr}}} = \frac{2 \times 192 \times 29 \times 10^6}{0.347 \times 9.87} \left[\frac{97.4 \times 0.220}{4 \times 7.08 \times 10^6} + \frac{0.441 \times 0.00305 + 5.56 \times 0.000945}{1210} \right]$$

$= 3260 \times 10^6 \times 6.21 \times 10^{-6} = \underline{\underline{20,200}}^{\#}$

If no support from verticals

$\underline{\underline{P_{cr}}} = 3260 \times 0.757 = \underline{\underline{2,470}}^{\#}$

$\underline{\underline{\text{Maximum Observed Load}}} = \underline{\underline{8,580}}^{\#}$

Buck.

For the verticals as cantilever-type columns assuming axial load:

$$P_{cr} = \frac{\pi^2 EI}{4\ell^2}$$

$$= \frac{9.87 \times 29 \times 10^6 \times 9.45 \times 10^{-4}}{4 \times 113}$$

$$= 600^{\#} \text{ for } 3/8 \text{ in.-diameter members}$$

$$P_{cr} = \frac{305}{94.5} \times 600 = 1.940^{\#} \text{ for } \tfrac{1}{2} \text{ in.-diameter members}$$

Truss No.	Member	Total Stress just before Failure	Diameter	
2	$U_1 - L_1$	1800	1/2 in.	
2	$U_2 - L_2$	1170	3/8 in.	greater than P_{cr}
2	$U_3 - L_3$	745	3/8 in.	" " "
2	$U_4 - L_4$	448	3/8 in.	
1	$U_4 - L_4$	478	3/8 in.	
1	$U_3 - L_3$	825	3/8 in.	" " "
1	$U_2 - L_2$	1120	3/8 in.	" " "
1	$U_1 - L_1$	1690	1/2 in.	

Once members $U_2 - L_2$ and $U_3 - L_3$ became incapable of support,

$$W_{TL} = 0.441 \frac{EI_1}{h^3} \delta_4^2 + \frac{EI_2}{h^3} \delta_4^2 \quad 1.5$$

Then:

$$\frac{0.347\pi^2}{2\ell} P = \frac{\pi^4 E I}{4\ell^3} + 0.441 E \frac{I_1}{h^3} + 1.5 E \frac{I_2}{h^3}$$

$$\underline{P_{cr}} = 3260 \times 10^6 \left[0.757 \times 10^{-6} + 1.11 \times 10^{-6} + 1.17 \times 10^{-6} \right]$$

$$= 3260 \times 3.04 = \underline{9.920}\ \#$$

Without $U_1 - L_1$

$$P_{cr} = 3260 \times 2.28 = \underline{7,440}\ \#$$

APPENDIX II

The Use of SR-4 Strain Gauges

The SR-4 electric strain gauge, by reason of its low cost, makes possible extensive investigations such as that reported in the previous portions of this thesis. While certain aspects of the gauges have their disadvantages, at least to the author's mind, they proved to be dependable and quite easy to use.

While detailed instructions for application may be obtained from the manufacturer upon application, it is probably as well to discuss, briefly, the method as used in the course of this experiment. For convenience, they will be listed as numbered steps.

1. Remove all loose scale and foreign matter from the surface on which the gauges are to be placed. For members which are readily handled singly, this is best accomplished by the use of a bench-sander or some similar piece of equipment. In the case of a number of fixed members, as in a truss, it is convenient to use some portable type of sanding equipment. If a particular position of the gauge is desired, centre lines should be inscribed, lightly, on the prepared surface.
2. Measure the resistance of the gauge before application. This is readily done with a Wheatstone Bridge fitted with probing leads, which allow the gauge resistance to be measured before removal from the package.
3. Clean the surface of the member with acetone.

4. Apply a liberal coating of cement to the contact surface of the gauge. The cements recommended by the manufacturer are a special SR-4 cement or Duco Cement No. 5418. The latter was used in this case, but, since completion of the experiment, the author has had occasion to examine work being done with these gauges at the University of Saskatchewan. At that institution very satisfactory performance has been obtained with the use of ordinary Duco Household Cement which may be procured much more readily.
5. Apply the gauge to the surface previously prepared and work all excess cement from between the gauge and surface by applying gentle pressure with the fingers. Care must be taken to prevent formation of bubbles in the cement layer.
6. Apply approximately one pound pressure to the gauge until the cement has become sufficiently "tacky" to hold the gauge in place. Suitable clamps for this purpose must be devised to fit the member to which the gauge has been applied. Normally, a period of between one and two hours was found to be sufficient for the glue to attain what might be called an "initial set".
7. Allow the cement a period of, approximately, twenty-four hours to attain a proper degree of solidification.
8. Measure the resistance of the gauge. This reading should not differ from that obtained in Step No. 2 by more than one-half ohm.
9. Measure the leakage resistance (i.e. the resistance between the gauge wires and the member) using a "megger" or megohm recorder. This leakage resistance should be greater than fifty megohms.

10. Attach leads of either No. 18 or No. 20, single solid, insulated wire using resin core solder to make the connections.

11. A compensating gauge of the same type, and preferrably the same lot, should be applied to an unloaded member, of like material, in the same manner. This is to offset changes in resistance due to temperature change.

It is well to note that, in order to be truly effective, the length of the leads on the compensating gauge should be the same as that of the leads on the recording gauge.

For ordinary tests on single members, the time required for preparation of the specimen is such that mechanical gauges are more conveniently used. The ordinary mechanical gauge has the additional advantage that it can be inspected during the test to ensure that it is functioning properly. However, where multiple-member structures are to be tested the SR-4 gauge is convenient and, seemingly, very reliable. The convenience is particularly evident when one remembers that all gauges may be read from a central control station.

APPENDIX III

Preliminary Tests

Test No. 1

Object - To determine whether or not an SR-4 strain gauge of the A3 type can be fastened to a $\frac{3}{8}$-inch diameter rod in such a manner as to give a satisfactory recording of the strains produced, in the rod, by the application of load.

Procedure - Measuring and compensating gauges were prepared using the method discussed in Appendix II.

Huggenberger extensometers were applied to the rod on the same section as was the SR-4 gauge, but at positions ninety degrees around the rod from the centre line of the electric gauge.

Zero readings were, mistakenly, taken before the application of any loading, and the specimen was then loaded to 3,710 pounds in increments of approximately, 200 pounds.

The test was conducted on the Baldwin-Southwark hydraulic testing machine.

Observations - The resistances of the measuring gauge before and after application were, respectively 119.4 ohms and 120 ohms. The leakage resistance was greater than 50 megohms.

The remainder of the observed and computed data are given in Table I and the results are depicted in Figure 1.

It will be noticed that the result from the SR-4 gauge is shifted somewhat to the right, indicating greater strains, and has a slightly steeper slope.

Conclusions - Apparently, failure to apply an initial load resulted in excessive apparent strain being indicated by the SR-4 gauge under the first two increments of load. After this load, the slack was removed from the gauge and the results became more uniform.

The greater slope of the SR-4 result is probably accounted for by eccentricity in the loading, in a plane containing the SR-4 and perpendicular to the plane containing the Huggenberger extensometers. This is quite likely since the specimen was an ordinary length of $\frac{3}{8}$-inch structural steel rod, not a specially machined tension specimen.

Test No. 2

Repeat of Test No. 1

Procedure - The only change of procedure was that an initial loading was applied and practically removed before starting the test.

Observations - The data is compiled in Table 2 and depicted in Figure 2. While the results coincide quite well, a difference in slopes still exists.

Conclusions - Eccentricity still seems to exist. This is shown in Figure 3, where the two sets of results coincide quite well when those from the SR-4 are multiplied by an eccentricity coefficient of 1.037.

It was extremely difficult to maintain a constant load on the specimen during the interval necessary to read the gauges. Hence, with this indication of satisfactory performance of the gauges on small members, it was decided to use a more reliable test for accuracy.

A. III

TABLE No. 1

TENSION TEST No.1

Area of Specimen= 0.107 sq.in.
Strains-micro.in/in.
Unit Stress-psi.
Load-lbs.

Load	Huggenberger Left	Huggenberger Right	SR-4	Difference SR-4	Difference Left	Difference Right	Huggen-berger Average	Unit Stress
			Stage 4					
0	250	20	988	0	0	0	0	0
200	310	50	1170	182	60	48	54	1870
400	380	100	1280	292	130	80	105	3740
600	460	180	1349	361	210	160	185	5610
800	530	240	1412	424	280	220	250	7480
1000	600	310	1479	491	350	290	320	9350
1190	650	370	1532	544	400	350	375	11100
1310	700	420	1588	600	450	400	425	12200
1500	760	490	1662	674	510	470	490	14000
1700	850	550	1710	722	600	530	565	15900
1900	900	610	1780	792	650	590	620	17800
2100	960	680	1849	861	710	660	685	19600
2300	1020	740	1920	932	770	720	745	21500
2500	1090	810	1987	999	840	790	815	23400
2700	1180	880	2052	1064	930	860	895	25200
			Stage 6					
2700			82					
2870	1240	940	130	1112	990	920	955	26800
3110	1310	1010	219	1201	1060	990	1025	29000
3220	1370	1050	242	1224	1120	1030	1075	30100
3470	1450	1120	330	1312	1200	1100	1150	32400
3710	1530	1200	400	1388	1280	1180	1230	34700
			Stage 5					
30	300	20	240	268	50	0	25	281

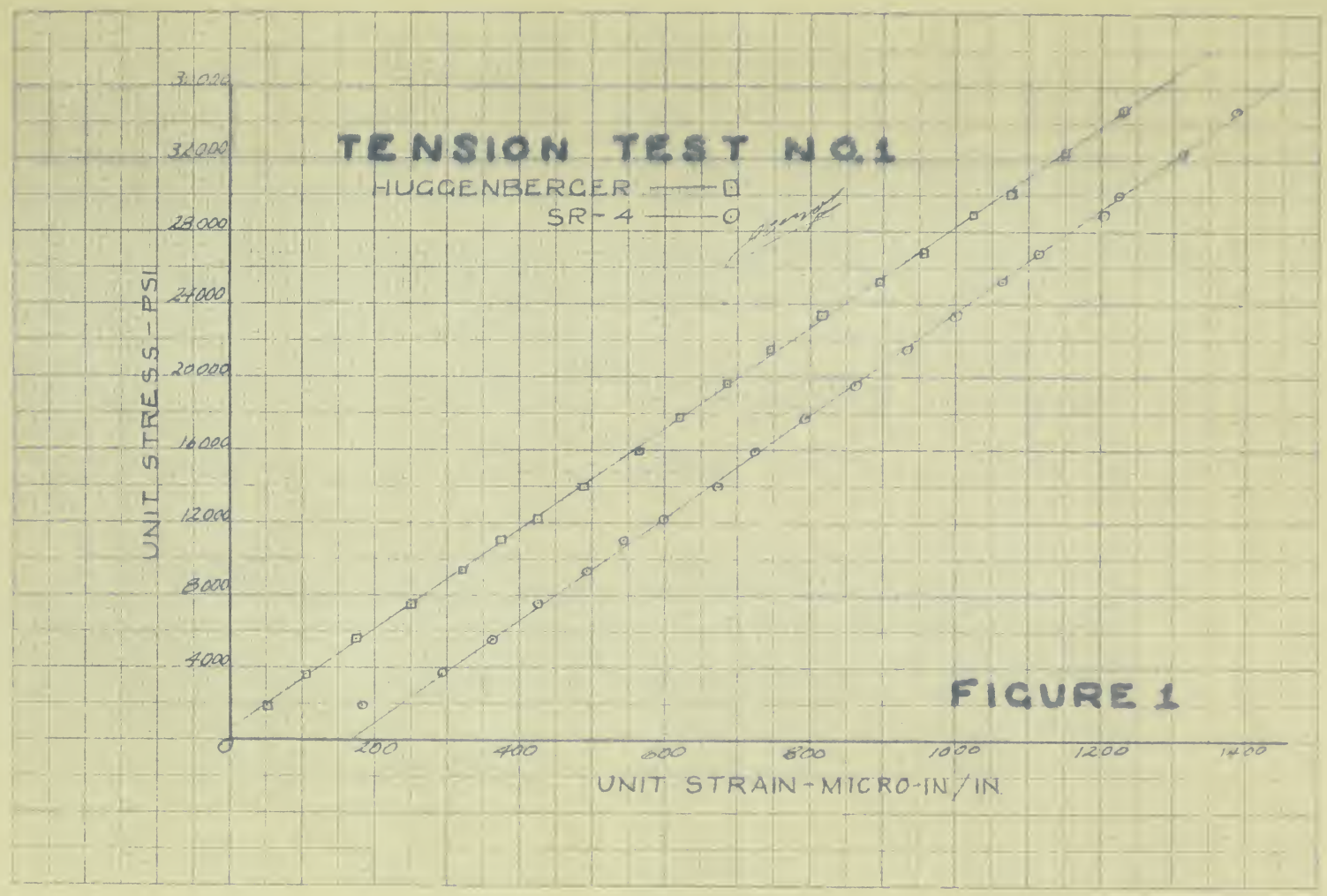

TENSION TEST NO. 1
HUGGENBERGER
SR-4
UNIT STRESS - PSI
32000
28000
24000
20000
16000
12000
8000
4000
0
200
400
600
800
1000
1200
1400
UNIT STRAIN - MICRO-IN/IN

FIGURE 1

A. III

TABLE No.2

TENSION TEST No.2

Area of Specimen= 0.107 sq.in.
Strains-micro.in/in.
Unit Stress-psi.
Load-lbs.

Load	Huggenberger Left	Huggenberger Right	SR-4	Difference SR-4	Difference Left	Difference Right	Huggen-berger Average	Unit Stress
			Stage 5					
0(30)	440	40	359	0	0	0	0	280
210	480	80	412	53	40	40	40	1960
400	540	140	463	104	100	100	100	3740
590	610	210	532	173	170	170	170	5510
790	670	270	592	233	230	230	230	7380
980	740	340	652	293	300	300	300	9160
1200	810	410	712	353	370	370	370	11200
1390	870	470	772	413	430	430	430	13000
1600	940	540	832	473	500	500	500	15000
1810	1000	600	895	536	560	560	560	16900
2000	1060	660	962	603	620	620	620	18700
2100	1090	690	992	633	650	650	650	19600
			Stage 6					
2100			29					
2390	1190	790	118	722	750	750	750	22400
2580	1250	850	182	786	810	810	810	24100
2790	1320	920	241	845	880	880	880	26100
3020	1390	990	315	919	950	950	950	28200
3210	1450	1050	369	973	1010	1010	1010	30000
3460	1520	1110	449	1053	1080	1070	1075	32300
3160	1440	1020	358	962	1000	980	990	29600
2350	1180	760	109	713	740	720	730	22000
			Stage 5					
2350			1072					
1940	1050	630	946	587	610	590	600	18100
1520	920	510	815	456	480	470	475	14200
1000	750	340	652	295	310	300	305	9350
530	600	180	500	141	160	140	150	4950
0	450	20	360	1	10	-20	-5	0

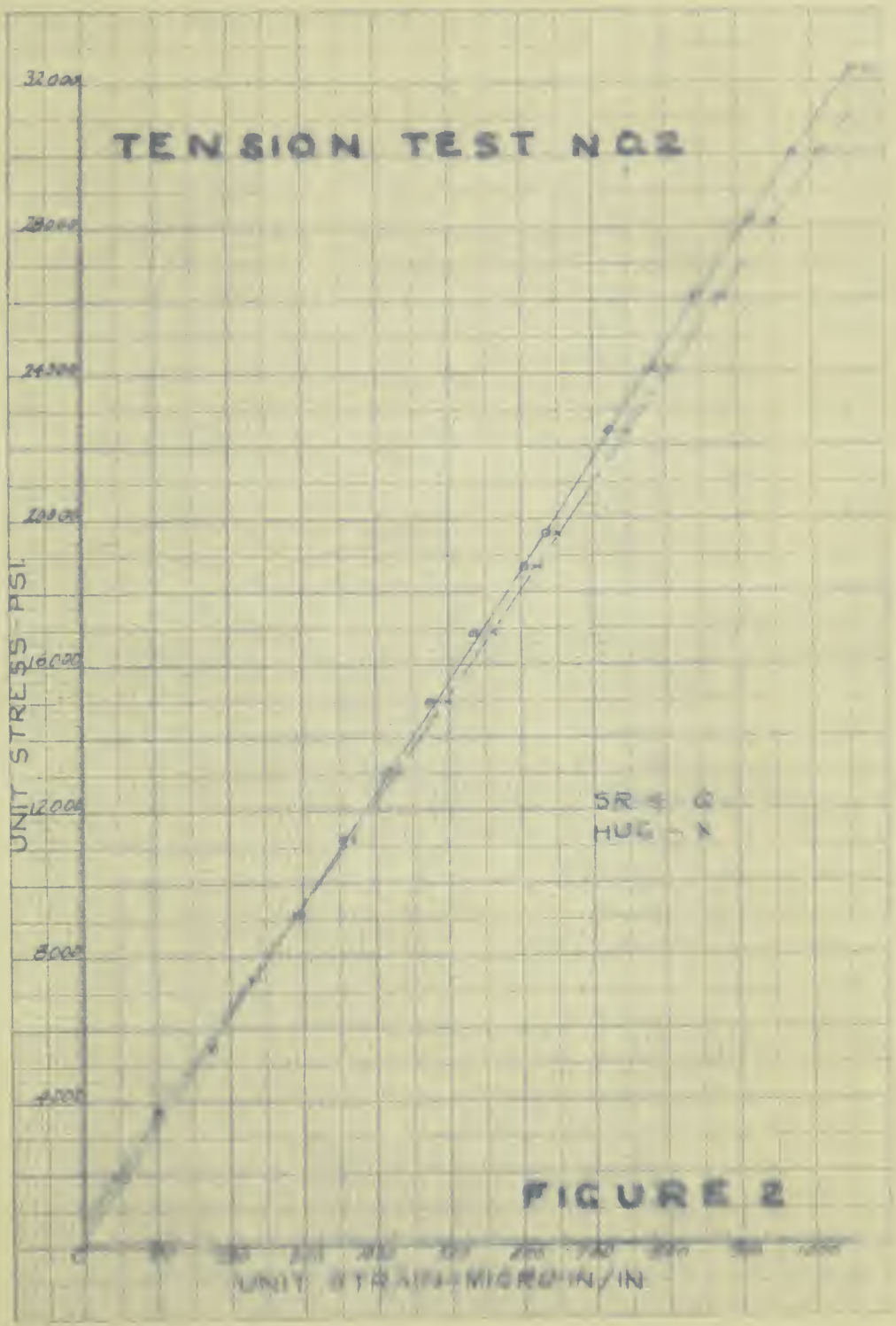

TENSION TEST NO 2
UNIT STRESS - PSI.
32000
28000
24000
20000
16000
12000
8000
4000
0
SR = G
HUG = X
UNIT STRAIN - MICRO-IN/IN

FIGURE 2

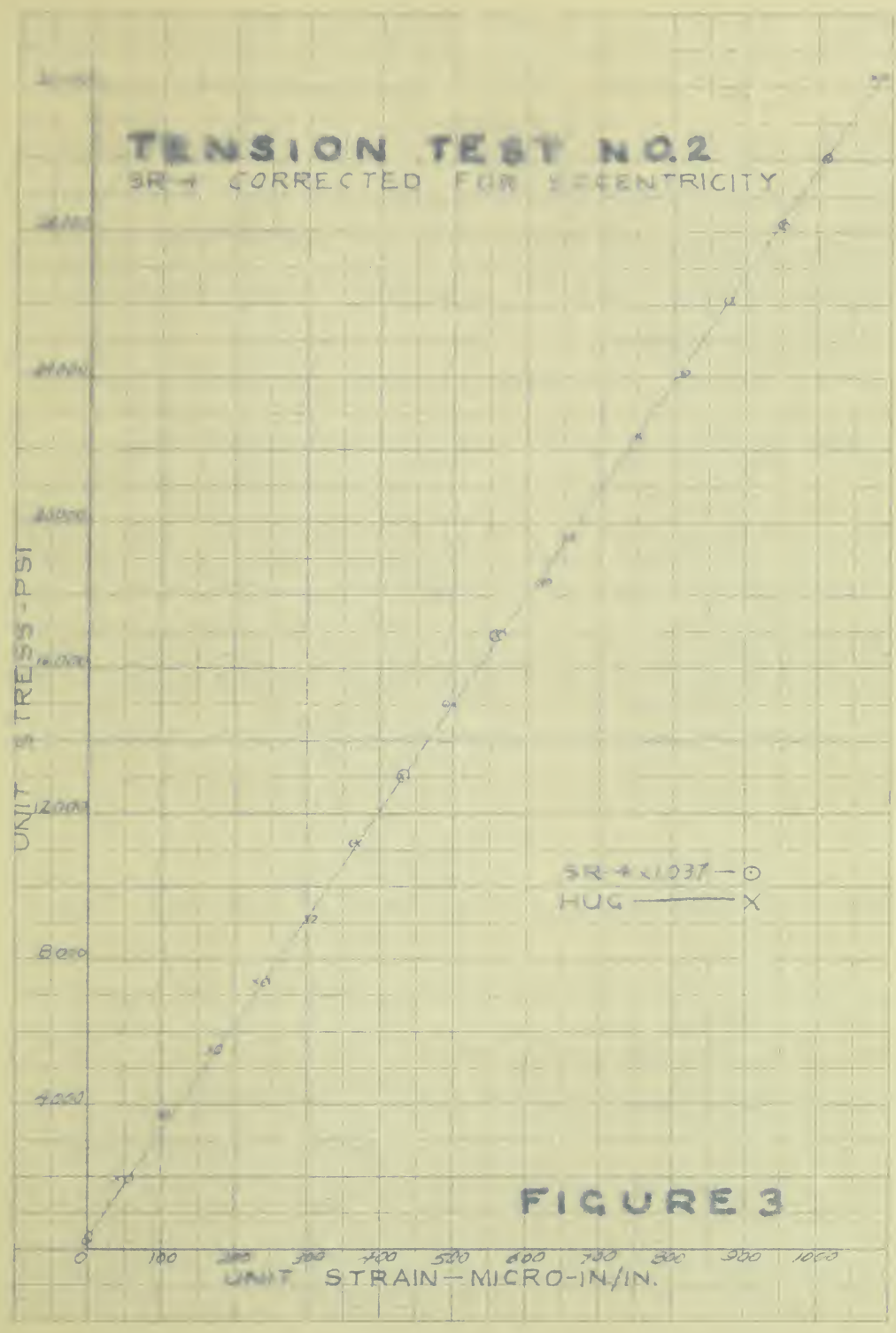

TENSION TEST NO.2
SR-4 CORRECTED FOR ECCENTRICITY
UNIT STRESS-PSI
SR-4 x1037 — ⊙
HUG ——— X
0
100
UNIT STRAIN—MICRO-IN./IN.

FIGURE 3

Test No. 3

Object - To determine the accuracy of SR-4 gauges by means of a constant load test, i.e. a dead load test.

Procedure - Gauges of the A1 type were applied to a piece of 1 inch x $\frac{1}{2}$ inch mild steel bar 2 feet long. These gauges were spaced approximately 14 inches from one end of the bar. At this end, a ball and socket connection was made for purposes of carrying a weight pan. This gave a good approximation of a concentrated or point load. The other end of the member was clamped, thus forming a cantilever beam with $\frac{1}{2}$ inch depth.

An initial load was applied and removed and then the member was loaded in four increments of approximately ten pounds each.

Observations - The results are shown in Table 3 and Figure 4. The value of 0.0035 inch used as a correction for the distance from the neutral axis to the centre of the strain gauge is the value quoted by the manufacturer as the average thickness of glue and paper beneath the gauge wires.

Conclusions - The accuracy of the gauges is perfectly satisfactory.

Test No. 4

Object - To determine the effect, on readings, of straining the compensating gauge.

Procedure - Using the bottom gauge on the cantilever beam as a compensating gauge, it was possible to apply an initial and constant strain to it before starting the tension test. A second tension specimen, fitted with Type A1 gauges, was prepared and used.

Observations - The results are shown in Table 4 and Figure 5.

Conclusions - While the balance readings (i.e.zero readings) were decreased by applying initial compression to the compensating gauge, no very noticeable change of results was obtained.

A. III

TABLE No.3

CANTILEVER BEAM TEST

Area of Specimen = 0.508 sq.in.
I of Specimen = 0.0102 in⁴
y of Specimen = 0.2456 in.
y for SR-4's = 0.2491 in.

Moment Arm-Top = 13.86 in.
Moment Arm-Bot.= 13.78 in.
Loads-lbs.
Stresses-psi.
Strains-micro-in./in.
Bending Moments-in.-lbs.

PART I - OBSERVED

Load	Readings Top	Readings Bottom	Strain Top	Strain Bottom
	Stage 6	Stage 6		
0	380	583	0	0
9.36	488	476	108	-107
19.5	609	353	229	-230
29.6	728	237	348	-346
39.4	842	120	462	-463
0	378	580	-2	-3

PART II - COMPUTED

Load	B.M.	f	Strain at top fibres	Strain at SR-4
		Top		
0	0	0	0	0
9.36	130	3130	108	109
19.5	270	6530	225	228
29.6	411	9930	342	347
39.4	546	13200	455	461
		Bottom		
0	0	0	0	0
9.36	129	3120	107	108
19.5	269	6490	224	228
29.6	409	9880	340	345
39.4	543	13100	452	458

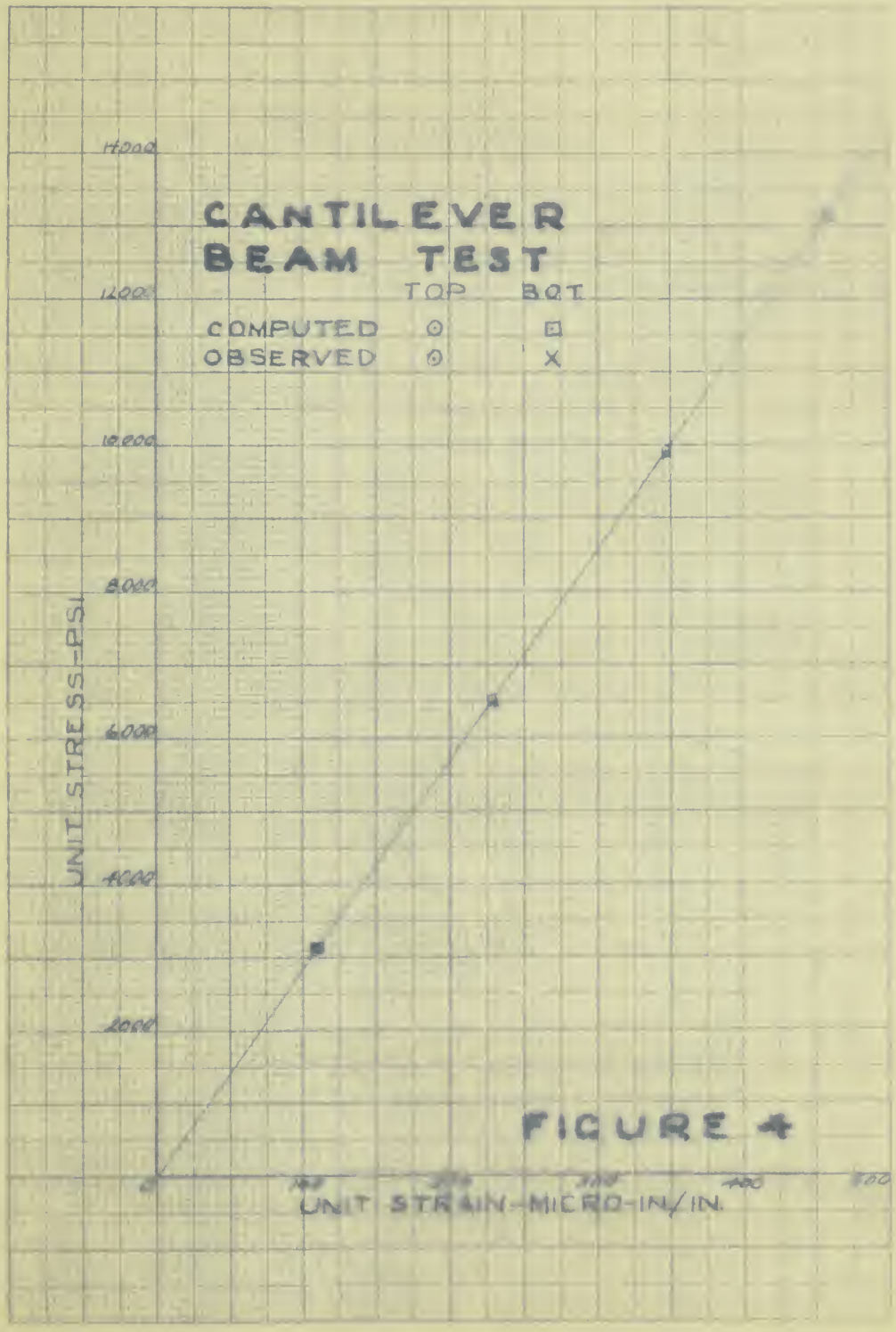

CANTILEVER
BEAM TEST
TOP
BOT
COMPUTED
OBSERVED
UNIT STRESS-PSI
UNIT STRAIN-MICRO-IN/IN.

FIGURE 4

A. III

TABLE No. 4

COMPENSATING GAUGE TEST

Loads-lbs.
Strains-micro.in/in.

Load	Readings Left	Readings Right	Difference Left	Difference Right	Strain (Average)
	1. Cantilever Unloaded				
	Stage 5	Stage 7			
10	440	162	0	0	0
430	563	233	123	71	97
870	670	343	230	181	206
1210	750	402	310	240	275
1580	832	490	392	328	360
1990	922	581	482	419	451
2400	1020	669	580	507	544
2820	1116	768	676	606	641
3200	1203	848	763	686	725
3600	1291	937	851	775	813
4040	1397	1030	957	868	913
15	471	141			
	2. Cantilever Loaded to 58.2 lbs.				
	Stage 4	Stage 6			
15	822	471	0	0	0
380	893	563	71	92	82
830	993	670	171	199	185
1360	1110	788	288	317	303
1710	1198	868	376	397	387
2020	1260	943	438	472	455
2450	1362	1029	540	558	549
2960	1480	1141	658	670	664
3190	1532	1190	710	719	715
3630	1643	1290	821	819	820
4010	1720	1359	898	888	893
20	822	477			
	3. Cantilever Loaded to 29.6 lbs.				
	Stage 5	Stage 6			
20	160	790	0	0	0
500	250	918	90	128	109
1030	370	1038	210	248	229
1540	489	1149	329	359	344
2000	590	1249	430	459	445
2520	712	1361	552	571	567
3070	838	1472	678	682	680
3530	948	1581	788	791	790
4010	1050	1692	890	902	896
20	153	792			

A.III

TABLE No. 4 (Cont'd)

COMPENSATING GAUGE TEST

Loads-lbs.
Strains-micro.in/in.

Load	Readings Left	Readings Right	Difference Left	Difference Right	Strain (Average)
		4. Cantilever Unloaded			
	Stage 5	Stage 7			
20	468	150	0	0	0
490	561	269	93	119	106
1060	689	403	221	253	239
1510	798	503	330	353	342
2040	910	613	442	463	453
2510	1019	719	551	569	560
3050	1141	843	673	693	683
3500	1242	933	774	783	779
3970	1352	1038	884	888	886
20	461	152			
		5. Cantilever Loaded to 19.5 lbs.			
	Stage 5	Stage 6			
20	259	903	0	0	0
440	338	1012	79	109	94
1090	489	1159	230	256	243
1550	591	1263	332	360	346
2180	738	1397	479	494	487
2830	888	1540	629	637	633
3030	937	1582	678	679	679
3500	1040	1689	781	786	784
4030	1162	1802	903	899	901
20	252	907			

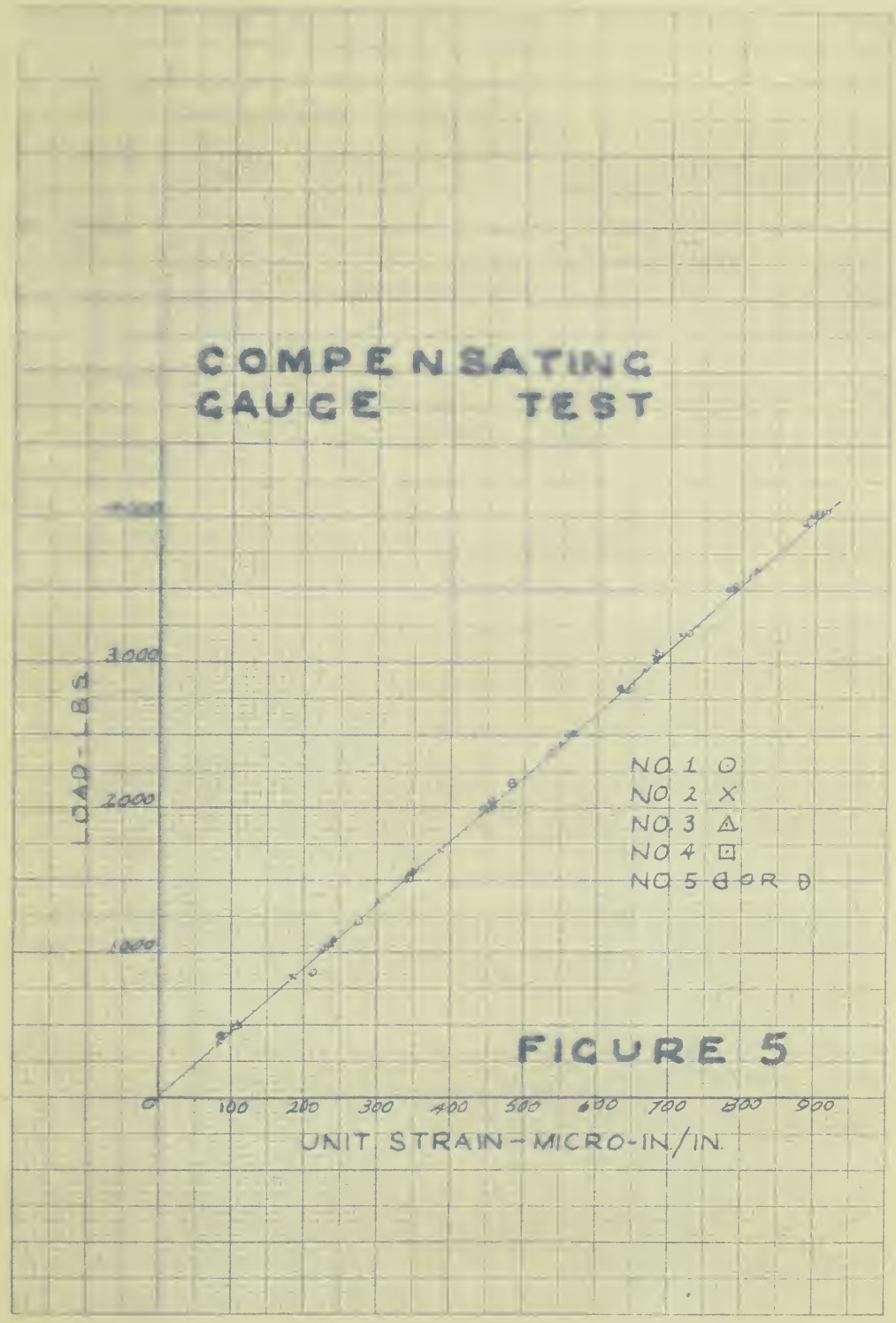

COMPENSATING
GAUGE TEST
LOAD - LBS.
3000
2000
1000
NO. 1 O
NO. 2 X
NO. 3 △
NO. 4 □
NO. 5 ⊖ OR θ
0
100
200
300
400
500
600
700
800
900
UNIT STRAIN - MICRO-IN./IN.

FIGURE 5

APPENDIX IV

Suggested Improvements

Several problems which arose during the experiment might have been circumvented by modifications of the equipment and the arrangement thereof.

1. Any further investigations should be carried out on trusses of more accurate and controlled construction, employing members of a slightly heavier and, preferrably, square or rectangular cross-section. Such a truss, with more careful joint construction, would permit of more rigorous analyses of the secondary stresses.
2. At least one truss should be fitted with gauges throughout its entirety to enable a "joint by joint" analysis of the results. This would greatly facilitate the analysis of results such as those obtained from the top chord.
3. The reaction supports should include a scale arrangement of some nature in order that the reactions could be determined accurately.
4. One reaction should be made as nearly frictionless as possible. This would improve the results obtained from the top chord.
5. An arrangement of loading whereby the load was applied to the truss at the panel points would facilitate the analysis.

With these modifications, it is probable that even the "locked-in" stresses could be, at least partially, evaluated.

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